Feasibility Report Appendixes

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American River Watershed Investigation, California

VOLUME 3 - APPENDIX M

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FEASIBILITY REPORT

LIST OF APPENDIXES

American River Watershed Investigation, California

APPENDIX M

Geotechnical Investigations

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

DOCUMENTATION REPORT

APPENDIX M

GEOTECHNICAL INVESTIGATIONS

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 1

GEOTECHNICAL BASIS OF DESIGN
LEVEE DESIGN REQUIREMENTS
FOR
NATOMAS EAST MAIN DRAIN, ARCADE CREEK, DRY CREEK, NATOMAS CROSS CANAL, SACRAMENTO RIVER, AND YOLO BYPASS

JULY 1989

PREPARED BY

SOIL DESIGN SECTION
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GEOTECHNICAL BASIS OF DESIGN

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

GEOTECHNICAL BASIS OF DESIGN
AMERICAN RIVER WATERSHED INVESTIGATION
RAISING OF EXISTING AND CONSTRUCTING NEW LEVEES

SCOPE

This Basis of Design's purpose is to address design aspects of those measures which include increasing the heights of approximately 114 miles of existing flood control levees and construction of approximately 9 miles of new levees along the east levee of the Natomas East Main Drain north of Dry Creek, the north bank of Dry Creek and north and south levees of Arcade Creek upstream of Del Paso Blvd. (Figure 1). No new geotechnical explorations were performed for this study. Conclusions and recommendations for the various levee reaches are based on the findings reached by the recent A/E investigations as well as past performance of the levees. For the purpose of the study the levees were divided into five separate reaches with similar soil and geographical conditions. General design conditions common to all the reaches are discussed, followed by reach specific information with regard to available exploration and laboratory data, existing levee and foundation conditions, and preliminary recommendations for each reach.

PREVIOUS STUDIES

Relatively minor geotechnical investigations and laboratory testing was accomplished on the flood control levees prior to construction or levee modification prior to adoption into the flood control system in the early to mid-1900's. However, following the February 1986 flood, five major A/E investigations with extensive geotechnical explorations and laboratory testing were performed to assess the present day condition of the levees (Appendix A). Therefore, only the post-1986 explorations are used in this study. The locations of these explorations are shown on Figure 1.

GENERAL DESIGN CONSIDERATIONS

The following paragraphs provide a general discussion of design considerations. Site specific considerations are covered under each reach.

Levee Stability

Prior to final design to raise or construct new levees, additional explorations and laboratory testing will be necessary.

Since explorations have already been performed on most of the existing levees, the additional explorations will concentrate mainly on borrow materials and foundation conditions of new levee alignments. Soil parameters including compaction and shear strength will be required for analyzing levee stability for the end of construction and steady seepage condition. Levees will be adequately designed to meet the stability requirements of EM1110-2-1913. Standard levee geometry used throughout the flood control system have generally provided satisfactory performance. Therefore, for the purpose of this study, the standard levee geometry is suggested for most levee reaches. That geometry includes levee slopes of 1V on 3H waterside and 1V on 2H landside with crest widths of 20 feet. For most of the levees requiring raising, the minor increases in levee height will, in general, have only a small overall effect on levee stability. However, this should be verified during the design studies. Information obtained from the previous investigations will be used to the extent possible in evaluating levee stability.

Erosion Potential

Where levee raising occurs over the landside slope, there will be no change in the existing erosion potential. Where raising will require levee construction over the waterside slope, a determination of the erosion potential of the borrow material and the need for slope protection will be required. Hydraulic considerations will also be considered in determining the need for slope protection for the various reaches. The requirements for erosion potential are discussed in Chapter 4 of this appendix and in Chapter 1 of Appendix N.

Liquefaction Potential

The levees along the Sacramento River are constructed primarily of sand and silty sand. These materials are typically loose and when saturated are susceptible to liquefaction during a large earthquake. Liquefaction results in a loss of soil strength due to load transfer from intergranular contact of soil particles to pore water. This can possibly lead to total levee failure or enough levee vertical displacement to cause overtopping of the levee. However, in order for liquefaction to occur, a simultaneous earthquake of large magnitude must occur at the same time as a major flood. Therefore, although the levees along the Sacramento River are susceptible to liquefaction, the probability for an earthquake occurring at the same time as a major flood is considered extremely remote.

Source of Construction Materials

Borrow sites must be identified by the local sponsors for future design studies. This should be accomplished prior to the design studies to facilitate field exploration and laboratory testing requirements necessary for design. It is most likely that

land adjacent to the existing levees and in the creeks or bypasses will be identified for borrow. Therefore, materials that will be used for raising will be very similar to the existing levee materials. Borrow sources should be located no closer than 75 feet to the levee toe. If the levees along the Sacramento River require raising, materials will likely be silt and sandy silt. Other borrow areas away from the Sacramento River, ie. Arcade Creek, Dry Creek, and the Bypasses will likely include finer grained materials such as clay, silty clay, or sandy clay.

LEVEE REACHES

Left Bank Sacramento River from Natomas Cross Canal to the Natomas Main Drainage Canal

Explorations and Laboratory Testing - Two exploration programs were conducted on this reach of levee following the February 1986 flood. The first exploration program (Ref. 1) was performed in conjunction with PL84-99 investigations north of I-5 along Garden Highway, in a reach of the levee where extensive levee damage occurred during the flood. These explorations consisted of eleven 8-inch hollow stem auger borings (DH-86-1 thru DH-86-11). Each boring was drilled from the levee crown to depths ranging from 30 to 31 feet. The second program (Ref. 2) was performed under the Sacramento River Systems Evaluation and was completed in July 1987. Explorations along Garden Highway under this program consisted of nineteen 8-inch hollow stem auger borings (DH-87-1 thru DH-87-15 and DH-87-37 thru DH-87-40). Majority of these borings were drilled through the levee crown. Borings DH-87-37 thru DH-87-40 were drilled adjacent to the levee landside toe. Standard penetration testing was performed on all borings except for intervals where undisturbed samples were taken for laboratory testing. Since the levee material in this reach is primarily clean loose sand, the only undisturbed samples taken were from the foundation where silt and clay soils are found.

Laboratory testing included gradation analyses, Atterberg Limits, and triaxial shear strength tests on undisturbed fine grained foundation samples. Laboratory test results are summarized in Table 1.

Existing Levee and Foundation Conditions - The levee crest width along this reach varies from about 25 to 45 feet and averages around 30 feet. The levee height ranges from about 10 to 20 feet with landside and waterside slopes of 1V on 2H and 1V on 3H respectively. The levee materials consists predominantly of very loose, fine to medium grained silty sand and sand. Foundation materials in this reach are more variable and consist mainly of fine grained silty sand, sandy to clayey silt and silty clay materials. Standard penetration tests and laboratory gradation tests indicate the sandy levee soils are poorly graded and the foundation silt and clay soils encountered are typically of firm consistency.

Design Considerations -

Levee Stability - Based on the geotechnical evaluations in this 18-mile reach, this levee is presently considered unstable during sustained flood conditions. However, remedial repairs are presently planned. The proposed repair will incorporate a sloping and horizontal drain and a stabilizing berm of approximately 7 feet in height and 12 feet wide. Any raising of the levee could be accomplished by constructing over the landside slope and stabilizing berm. To ensure seepage collected by the sloping drain is transmitted safely through the levee, the horizontal drain would also have to be extended. If this is accomplished raising of the levee will have very little affect on stability of the levee.

Seepage - Seepage will not be affected by any increase in levee height along this reach. As evidenced during the February 1986 flood, seepage is prevalent during high water conditions. After repairs are made, quantity of seepage will not significantly change. However, the seepage through the levee will be controlled.

Settlement - Any settlement of the raised levees will be small since the levees were constructed primarily of sandy soils. Most of any settlement that does occur, will occur during construction. Foundation settlement should be negligible for increases of levee heights of less than about 5 feet.

Preliminary Recommendations - Increasing the levee height along this reach can be accomplished by raising the levee crown with levee construction on the landside of the levee over the proposed landside berm (Figure 2). If the levee is required to be raised more than about 2 feet, the future landside berm will be required to be extended approximately 8 feet landward. This will be necessary to maintain the upper portion of the landside slope no steeper than 1V on 2H. In addition, a paved highway exists on the levee and therefore will require removal and replacement of the road surfacing.

Source of Construction Materials - Majority of the materials required for construction will probably be obtained in the interior of Reclamation District 1000. It is anticipated that several borrow sources will have to be identified within the interior of Reclamation District 1000 in order to minimize haul distances of levee material. The materials will likely vary from sand to sandy silt and will be acceptable for the purpose of raising the levee along this reach. If extension of the landside berm is necessary, a commercial aggregate source will be required to extend the horizontal drain.

Natomas Cross Canal, East Side Canal, East Main Drainage Canal and South Levee of Reclamation District 1000 to the Natomas Main Drainage Canal

__Explorations and Laboratory Testing__ - Two exploration programs were undertaken in this reach following the February 1986 flood. In the first of these (Ref. 2), borings DH-87-16 through DH-87-36 (crown borings) and DH-87-41 through DH-87-46 (landside toe borings) were drilled. In a re-evaluation study (Ref. 3) of the south levee of the Natomas Cross Canal, additional explorations included borings DH-88-16A through DH-88-21B with companion toe trenches. The borings were drilled with an 8-inch hollow-stem auger with continuous standard penetration testing except where undisturbed samples were collected for laboratory testing. The trenches were dug with a backhoe along the landside toe of the south levee of the Natomas Cross Canal to evaluate conditions which have historically caused instability in the vicinity just west of Highway 99. A third exploration program (S1 through S7) was performed during a geotechnical investigation of the north levee of the Natomas Cross Canal by a private consultant for Reclamation District 1001 (Ref. 4).

Laboratory testing along this reach of the study consisted mainly of gradation and Atterberg Limit testing with some triaxial shear strength testing of representative undisturbed 3-inch Shelby tube samples. The results of the laboratory tests are summarized in Table 1.

__Existing Levee and Foundation Conditions__ - The levee along the Natomas Cross Canal varies in height from approximately 11 to 20 feet, with the crest width varying from about 25 to 60 feet. The height of the levee along the East Main Drainage Canal and south levee of R.D. 1000 varies from about 5 to 20 feet and the crest widths vary from 22 to 55 feet. The levee slopes in this reach are 1V on 2H on the landside and 1V on 3H on the waterside. The materials along the north and south levee of the Natomas Cross Canal are predominantly fine-grained silts and clays of low to medium plasticity with a firm to stiff consistency. The geotechnical investigations along the south levee of the Natomas Cross Canal revealed a weak silt seam 2 to 3 inches thick in the area just west of the Highway 99 crossing of the south levee. This layer of material exists approximately 3 to 4 feet below the foundation. It was concluded that this seam was partially responsible for landside slope instability in February 1986 in the area just west of Highway 99. This same foundation condition likely contributed to slope failures in this same area in 1955 and 1983.

Levee materials along the East Main Drainage Canal are similar with some occurrences of clayey and silty sand within the levees. A hardpan (cemented) layer of very stiff to hard silt to clay material exists in the foundation throughout most of this reach. This layer is typically 5 feet deep along the Natomas Cross Canal and up to at least 15 feet thick along the East Main Drainage

Canal. The hardpan layer found in most of this study area was not encountered along the south levee of R.D. 1000. Instead, the foundation materials in that area vary from stiff silty clay to loose to medium dense clayey sand.

The East Side Canal extends about 4.5 miles above the eastern extent of the Natomas Cross Canal. There is presently no geotechnical information available on this reach. For the purpose of this study, it is assumed that material in this reach is similar to that of the majority of the soils in this study reach. Explorations will be required along this reach prior to designing levee modifications.

Design Considerations -

Levee Stability - The geotechnical evaluations made of the levees in this reach concluded that except for a portion of the south levee of the Natomas Cross Canal, the levees are generally stable. Therefore, since the increases in levee heights will be small compared to the existing levee height, instability of the levees is unlikely. The present Sacramento River Systems Evaluation plan is to relocate the irrigation ditch adjacent to the south levee of the Cross Canal as recommended in the re-evaluation study of the south levee (Ref. 3). Once this is accomplished, raising of any of the levees in this study area should not cause any stability problems.

Seepage - Because of the generally low permeability soils along these levee reaches, seepage is not a problem. Any increase in levee heights will have very little if any affect on the overall potential for seepage during high water conditions.

Settlement - The levee and foundation materials in this reach are primarily silt and clay. Therefore, there will be some, although relatively minor post-construction settlement. An estimate of this settlement will be dependent on the amount of increase in levee height required. Laboratory consolidation tests will be required prior to design to determine the settlement characteristics of the borrow and foundation soils in this reach.

Preliminary Recommendations - The levees along most of this reach can be raised without jeopardizing levee stability. The only area that may require special consideration is the Natomas Cross Canal. Present remedial repair plans along this reach call for the relocation of the drainage ditch adjacent to the landside levee toe along a 1.3 mile reach west of Highway 99. If the levees are to be raised along this reach, in addition to relocating the irrigation ditch along the entire length of the levee, it is recommended that the landside slopes be modified to 1V on 2.5H. The landside slope can be maintained at 1V on 2H along other portions of this study reach. The levee along approximately the southern two miles portion of the East Main Drainage Canal may require construction on the canal side due to residential development in this area. In addition, a paved

highway exists on the levee crown along all but the Natomas Cross Canal and the East Side Canal. Therefore, levee raising would also require removal and reconstruction of the paved road surface.

Source of Construction Materials - Borrow material for levee raising along this reach will be identified by the local sponsor. However, it is likely that the Natomas Cross Canal and the East Main Drainage Canal will be identified for borrow material. Based on existing exploration data, these soils are fine grained silt and clay and will be adequate for the purpose of increasing the height of the levees.

Dry Creek, Arcade Creek, and East Bank Levee of the East Main Drainage Canal

Explorations and Laboratory Testing - Explorations in this reach were conducted in April 1987 (Ref. 5) and included a total of eleven 8-inch hollow-stem auger borings and continuous (except where undisturbed samples were taken) standard penetration. The borings, DH-13 through DH-20, were drilled to depths ranging from 20 to 45 feet.

Laboratory testing included gradation, Atterberg Limit testing and triaxial shear strength tests. Laboratory test results are summarized in Table 1.

Existing Levee and Foundation Conditions - Levees in this area are typically 7 to 18 feet in height with side slopes of 1V on 2H on the landside and 1V on 3H on the waterside. The east levee of the Natomas East Main Drain and levees on Arcade Creek upstream of Rio Linda Blvd. were constructed with a crest width of 12 feet. The Dry Creek levee and the Arcade Creek levee downstream of Rio Linda Blvd. were constructed with a crest width of 20 feet.
The levee and near surface materials in this area are generally comprised of firm to stiff, low plasticity sandy clay to clayey sand. In addition, explorations north of Arcade Creek consistently penetrated a foundation hardpan layer of sandy silt to silty sand 5 to 10 feet thick (N=25 to 57). Beneath this depth the foundation materials are generally comprised of stiff clayey silt to silty sand.

Design Considerations -

Levee Stability - The geotechnical evaluation performed for this reach concluded that the existing levees are stable and minor increases in heights should not jeopardize levee stability. Based on that evaluation and past performance, the levees in this reach could undoubtedly be raised up to at least 5 feet with minimal risk of levee instability. This will be verified during design using soil parameters obtained from borrow material sampling and testing. New levees constructed in this reach will be designed using the appropriate engineering criteria. For estimating purposes, the new levee geometry should tentatively be assumed to have 1V on 3H waterside and 1V on 2H landside slopes with a 20-foot crown.

__Seepage__ - Seepage is of very little concern in this reach. The levee and foundation materials are predominantly fine grained silts and clay and therefore relatively impervious to seepage during short term flood conditions. Therefore, instability due to piping and seepage induced slope failure are not considered to be a potential problem.

__Settlement__ - Some, although very little long term settlement can be expected in this reach depending on the increase in levee heights or heights of the new levees built. However. settlement of the foundation should be negligible since the foundation consists of very firm and cemented materials.

__Preliminary Recommendations__ - Raising of levees in this reach can be accomplished without affecting levee stability. New levee construction along the upper reaches of Arcade Creek and the north side of Dry Creek can be accomplished with relatively minor design problems. New levees should be constructed using the standard 1V on 2H landside and 1V on 3H waterside slopes and a 20-crown. It is anticipated that levee raising along Arcade and Dry Creek will be accomplished on the waterside due to existing residential development.

__Source of Construction Materials__ - As with other reaches borrow material for raising or construction of new levees will be identified by the local sponsor. Borrow material from the Natomas East Main Drain may be limited due to cemented silt and sandy silt deposits identified during explorations. It is anticipated that borrow sources will be identified on the waterside adjacent to the existing or new levees to be constructed.

Right Bank Levees of the Sacramento River from Verona to the Confluence of the American River

__Explorations and Laboratory Testing__ - Existing explorations along this reach is limited. The only exploration information available includes the 1986 PL84-99 boring (Ref. 6) at the north end of this reach (Boring Site 5) and borings and trenches at sites DH-25, 25.5, 26 along the right bank of the Sacramento River west of the confluence with the American River (Refs. 7 and 8). Laboratory testing includes gradation, Atterberg Limits and shear strength tests on both the levee and foundation materials. These explorations and laboratory data are available for use in the design of raising levees if required. Results of the available laboratory and field data along this reach is included in Table 1. At the present time, an A/E geotechnical evaluation of the right bank of the Sacramento River from Verona to the Sacramento Weir is being conducted as a part of the Mid-Valley flood control systems evaluation. This program includes an exploration program consisting of cone penetration testing, standard penetration

testing and undisturbed sampling for laboratory testing. This study, which will include engineering analyses with recommendations is scheduled for completion in late September 1989.

__Existing Levee and Foundation Conditions__ - Based on the presently available geotechnical data, the levee along this reach has a nominal 20-foot crest width with about 1V on 3H waterside and 1V on 2H landside slopes. These dimensions vary locally with slightly wider crests and flatter slopes. Available information at the northern and southern end of this reach indicates the levee and foundation conditions for this reach are very similar to the east bank levee of the Sacramento River (R.D. 1000). That is, very sandy levee soils constructed on soft to firm foundation silt and clay soils. More defined information on the levee conditions will be available following the Mid-Valley geotechnical evaluation.

__Design Considerations__ -

__Levee Stability__ - Levee stability in this reach is tentatively considered similar to that of the east bank levees. Only minor seepage and boil problems reportedly occurred along the northern portion of this reach during the February 1986 flood. However, the levee materials make this reach susceptible to damage during future flood events. The currently underway A/E evaluation will provide more information on levee stability in this reach. If this reach of levee is raised, it is presently envisioned that landside slope will have to be flattened to about 1V on 2.5H or internal drainage and or berms would be required in the design.

__Seepage__ - As discussed before, the sandy levee soils in this reach are considered susceptible to seepage and potential piping during sustained flood flows. Raising the present levees would not in itself increase seepage. However, increasing the level of flood protection equates to increased hydrostatic pressure head on the levees. Although this will likely cause increased seepage through the sandy levee, stability will not be decreased if the levee raising is properly designed.

__Settlement__ - Similar to the east bank levees, any post-construction settlement that occurs during raising of this reach of levee should be negligible. Since the levee is believed to be primarily sand any settlement will occur during construction. The composition of the levee material will be known more accurately following the A/E geotechnical evaluation.

__Preliminary Recommendations__ - Levee raising along the right bank of the Sacramento River can be accomplished using landside construction. Depending on the results of the ongoing A/E evaluation on this reach, seepage remedial measures may be required. If remedial measures are required, they would likely consist of flatter (1V on 2.5H) landside slopes or construction of a seepage berm and internal drain similar to that presently being

considered for the left bank levee. Levee raising will also require relocation of railroad tracks and reconstruction of the highway along portions of the levee crown.

Source of Construction Materials - Borrow sites for levee raising of this reach of levees will be identified by the local sponsor. Materials will likely be borrowed from nearby inland sources and can consist of almost any mineral type soil free of organics.

Sacramento and Yolo Bypass Levees from Knights Landing Ridge Cut to Putah Creek

Explorations and Laboratory Testing - Limited explorations have been performed in this reach. However, some explorations and laboratory testing of the east levee of the Yolo and south levee of the Sacramento Bypasses were performed during two separate studies. The first of these (Ref. 6) was performed on the Yolo Bypass levee north of the Sacramento Bypass in 1986 during PL84-99 work following February 1986 flood. A total of six borings were drilled through the levee crown in R.D.s 827 and 785 to a depth of 40 feet. The second study was performed under the Sacramento River Flood Control System Evaluation, Sacramento Urban Area (Ref. 7). These explorations included 7 auger borings with standard penetration tests drilled at the levee crown and at the landside toe ("A" borings). At present, a comprehensive geotechnical evaluation is being conducted for the Yolo Bypass levees by an A/E consultant for the Corps of Engineers. The purpose of the study is to evaluate the present condition of the levees. Results of this study will be available and be used for preliminary design of any levee raising that may be required in this reach.

Existing Levee and Foundation Conditions - Levee heights typically range from approximately 10 to 20 feet. Levee crown widths range from 20 to 29 feet, although the west levee crown widths are a nominal 12 feet wide. Based on available data obtained from the two A/E studies of the east bank of Yolo Bypass, the levee and foundation soils in this reach are primarily lean to fat clay with some organic content. The levee soils in the Yolo Bypass typically exhibit random cracking near the surface during the dry months of the year. Typically the soil swells in the winter months and the cracks close. Therefore, seepage is generally of little concern. However, cracks in the levee soils have resulted in potentially weaker zones within the levee and have been suspected of being associated with past instability.

Laboratory testing was performed on samples obtained during both studies discussed above. Testing included gradation, Atterberg Limits, and some unconsolidated undrained shear strength tests. Test results are included on Table 1.

Design Considerations -

Levee Stability - Levee instability has been an intermittent problem along the Yolo Bypass. Levee and landside slope failures have been a particular problem on the east levee north of the Sacramento Bypass. These failures typically originate near the adjacent landside irrigation ditch and progressively work into the levee crown. During the rainy season the medium to high plasticity clay soil in the levee becomes saturated and loses strength and overstresses the foundation. Any raising of the Yolo Bypass levees will require careful consideration of slope stability. It is anticipated irrigation ditches located near the landside levee toes will need to be relocated a minimum distance of 75 feet away from any levees that are raised. Based on frequent maintenance repairs of both waterside and landside slopes of various stretches of levee along the east levee of the Yolo Bypass, additional measures to insure levee stability may be necessary. This could include landside or waterside berms and/or flattening of the landside slope to 1V on 2.5H.

Seepage - Since both the levee and foundation soils in this reach are primarily clay, seepage has not been a problem. Materials used for levee raising will likely be of similar soils and therefore no future problem related to seepage is anticipated. Although some of the clay soils that were used to construct these levees are subject to cracking, the cracks seal during periods of rain prior to the Yolo Bypass being flooded.

Settlement - Some settlement of raised levees in the Yolo Bypass will occur. This is because the foundation soils are primarily soft clay and will undergo some consolidation upon loading. Therefore, laboratory consolidation testing should be performed prior to design. This data will be used in settlement calculations. If the increase in levee heights are less than the five feet as anticipated, post-construction settlement should be minimal.

Preliminary Recommendations - Since a geotechnical evaluation of most of this study reach is presently underway, the following recommendations should be considered preliminary. Levees along most portions of the reach could be raised on either side. However, landside construction would be preferred in terms of quantity of material required and since portions of the waterside slopes are presently protected with slope protection. As mention earlier, irrigation ditches along portions of this reach will be required to be moved and located no closer than 75 feet away from the levee toe.

Source of Construction Materials - Material for levee raising will likely be identified in the Yolo or Sacramento Bypass. These material borrow sources will have to be identified by the local sponsor and be tested by the Corps prior to the design to increase levee heights. Since compaction and general working conditions of

high plastic clay (CH) is difficult, sources of low to medium plastic clay (CL) should be identified. The Sacramento Bypass will likely be identified as a good source of sand and sandy silt borrow material.

APPENDIX A

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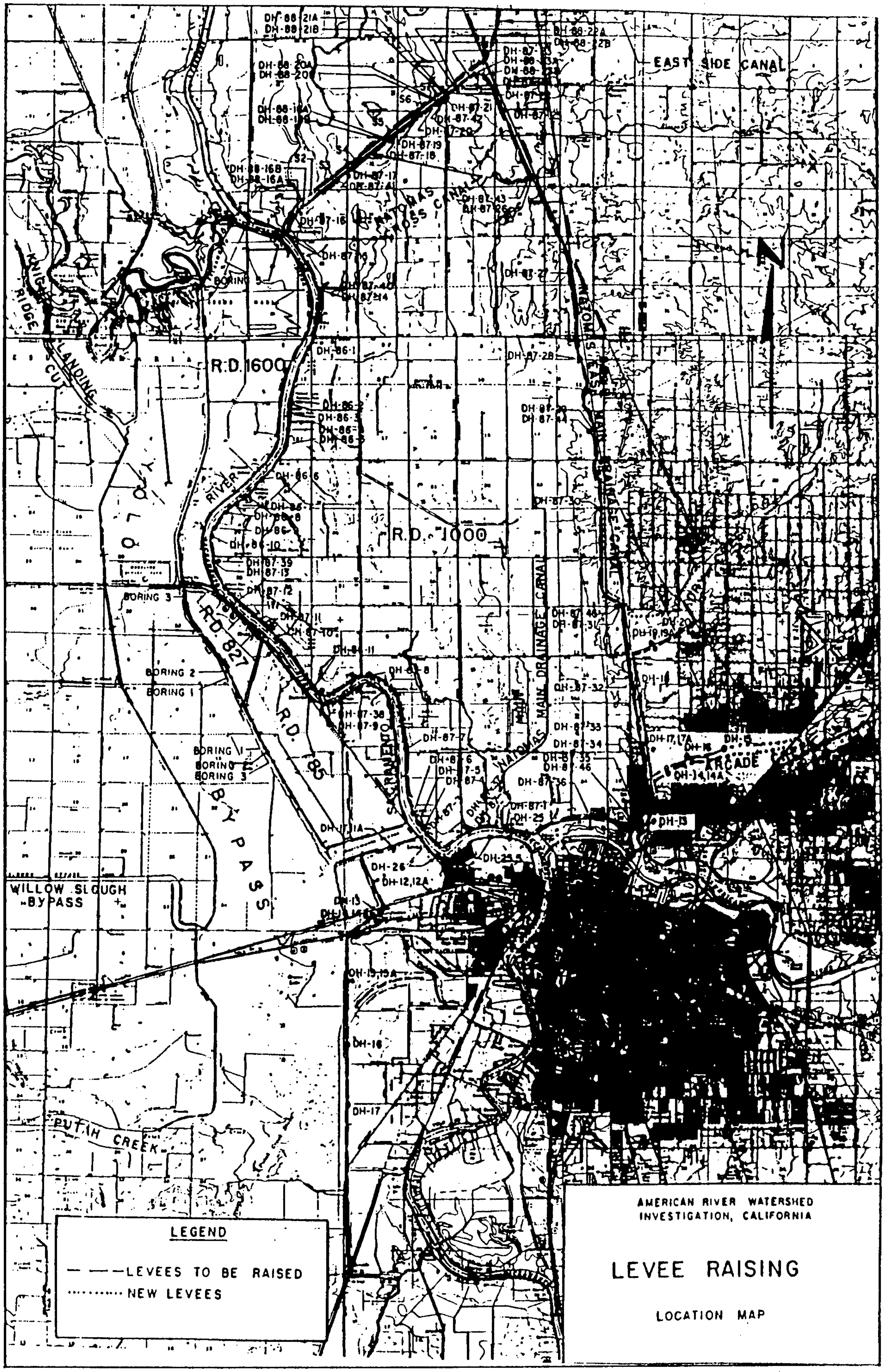
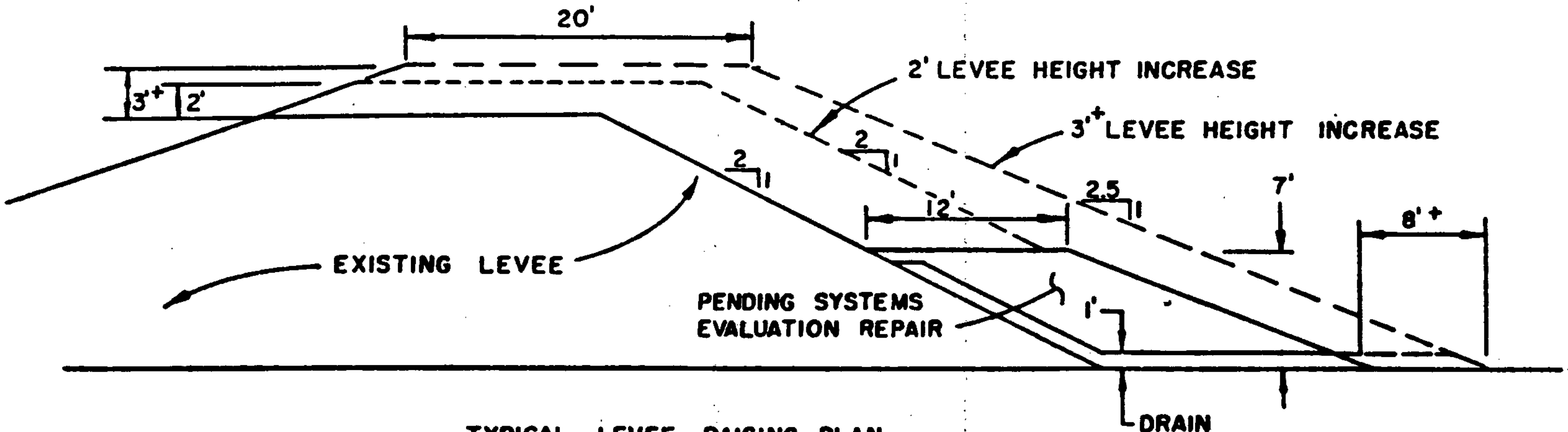
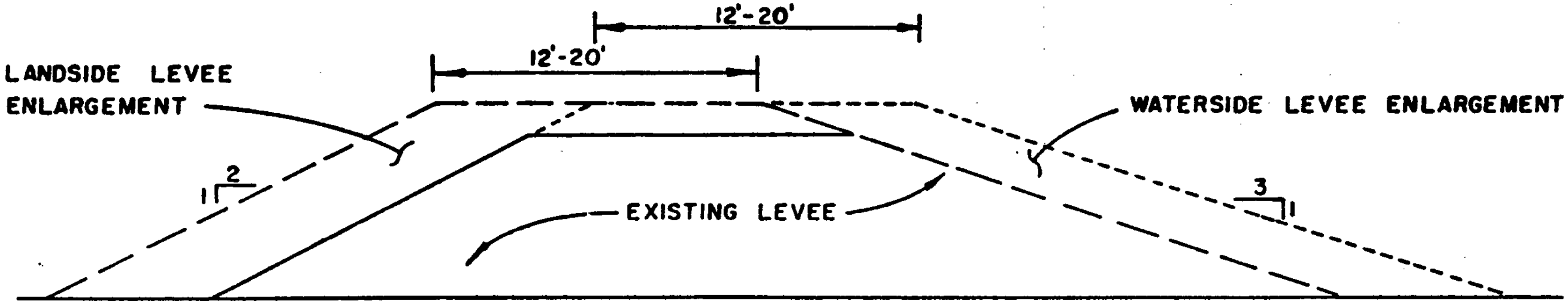

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA
LEVEE RAISING
LOCATION MAP
LEGEND
LEVEES TO BE RAISED
NEW LEVEES
EAST SIDE CANAL
R.D. 1600
R.D. 1000
R.D. 827
R.D. 785
YOLO BYPASS
WILLOW SLOUGH
BYPASS
PUTAH CREEK
KNIGHTS LANDING RIDGE CUT
SACRAMENTO RIVER
NATOMAS CROSS CANAL
NATOMAS MAIN DRAINAGE CANAL
ARCADE
BORING 1
BORING 2
BORING 3

FIGURE 1



TYPICAL LEVEE RAISING PLAN
(GARDEN HIGHWAY)



TYPICAL LEVEE RAISING SECTION

TYPICAL LEVEE RAISING ALTERNATIVES

FIGURE 2

TABLE 1 - SUMMARY OF EXISTING LABORATORY AND FIELD DATA

LEVEE REACH	PRIMARY SOIL CLASSIFICATION	SPT BLOW COUNT (N) Range (Avg.)	DRY UNIT WEIGHT (PCF) Range (Avg.)	MOISTURE CONTENT % Range (Avg.)	AVG. LAB. GRADATION GR	SA	FI	ATTERBERG LIMITS Range (Avg.) LL	PI	LABORATORY SHEAR TEST DATA
Lt. Bank Sac. River, Natomas Cross Canal to Natomas E. Drainage Canal										
Existing Levee	Loose Sand (SP) to Silty Sand (SM)	1-13 (7)	-	-	5	84	11	NP	NP	-
Foundation	Loose Silty Sand (SM) to Sandy Silt (ML)	2-26 (10)	70-98 (85)	7-51 (18)	0	26	74	27-58 (38)	5-25 (12)	Q=720-3660 (1800) psf
Natomas Cross Canal, E. Main Drainage Canal (NEMDC) and S. Levee of R.D. 1000										
Existing Levee	Firm to Stiff Sandy Clay (CL) and Sandy Silt (ML)	2-29 (10)	70-104 (92)	9-43 (22)	0	41	59	26-51 (41)	12-31 (21)	Q=1200-2340 (1800) psf R=15-22 (17) deg.,c=120-430(280)psf S=32-37 (34) deg.
Foundation	Similar w/some cementation	4-85 (35)	73-108 (95)	5-44 (22)	0	37	63	22-65 (41)	13-28 (21)	Q=2000-3900 (3400) psf R=15-20 (16) deg.,c=30-775(425)psf S=20-35 (27) deg.
Dry Creek, Arcade Creek, and E. Bank Levee of the NEMDC										
Existing Levee	Firm Sandy Clay (CL)	2-15 (6)	88-105 (97)	17-28 (23)	0	46	54	20-51 (36)	5-34 (17)	Q=1700-1800 (1750) psf
Foundation	Firm Sandy Clay (CL) to Clayey Sand (SC) w/some cementation	3-47 (17)	82-119 (104)	8-20 (14)	0	54	46	24-42 (34)	4-26 (13)	Q=1700-2700 (2100) psf
Rt. Bank Levee Sac. River, Verona to Confluence of the American River										
Existing Levee	Very Loose to loose Silty Sand (SM) to Sand Silt (ML)	2-22 (8)	69-80 (74)	1-3 (2)	0	96	4	NP	NP	-
Foundation	Firm Lean Clay (CL) to Silty Sand (SM)	2-12 (6)	71-80 (75)		0	28	72	30-45 (33)	6-21 (11)	Q=3300 psf R=11-12 deg., c=0-200 (100) psf S=24-28 (26) deg.
Sac. Bypass and the Yolo Bypass from Knights Landing Ridge Cut to Putah Creek										
Existing Levee	Soft to Stiff Lean to Fat Clay (CL to CH)	4-17 (9)	76-105 (94)	20-40 (27)	0	4	96	49-69 (57)	32-45 (38)	Q=600-2800 (2000) psf
Foundation	As above with organics	1-28 (9)	67-93 (84)	25-52 (36)	0	13	87	5-46 (33)	5-46 (33)	Q=800-1800 (1300) psf

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 2

STABILITY ANALYSIS FOR AMERICAN RIVER LEVEES

OFFICE REPORT

JULY 1988

PREPARED BY

SOIL DESIGN SECTION
GEOTECHNICAL BRANCH

OFFICE REPORT
LEVEE STABILITY
AMERICAN RIVER LEVEES

TABLE OF CONTENTS

TABLES

FIGURES

TABLE OF CONTENTS (CONT.)

APPENDICES

OFFICE REPORT
LEVEE STABILITY
AMERICAN RIVER LEVEES

PURPOSE

This report presents the results of a follow-up stability study of the American River levees. Following the February 1986 flood in Sacramento, an evaluation of the flood control levees protecting the Sacramento Metropolitan area was undertaken by the Sacramento District Corps of Engineers. The Architect-Engineering (A/E) firm, Wahler Associates of Palo Alto, California, was contracted to perform explorations along the American River levee system and provide a preliminary geotechnical evaluation of the levee conditions. That report, completed in September 1987 (Ref. 9), concluded that the American River levees are generally unstable and do not meet the Corps minimum stability factor of safety of 1.4. The field exploration methods used in that study (standard penetration tests - SPT) provides only an approximation of the soil strength and therefore conservative soil shear strength values were used in the analyses. In addition, a conservative value of 3 feet of freeboard was used throughout the study area. Actually, freeboard at a flood flow of 130,000 cfs, is a minimum of 4.9 feet throughout the study reach (see Table 2). The project was designed for a flow of 115,000 cubic feet per second (cfs) with a minimum freeboard of 5 feet. The A/E report recommended additional study to confirm the findings in the September 1987 report. This study was undertaken as a result of recommendations in the A/E report as well as to determine the need for remedial measures to insure stable levee conditions for the design flow of 115,000 cfs.

SCOPE

The levees in this study (See location map, Figure 1) extend on the right bank of the American River from about one mile upstream of Arden Way, downstream to the junction of the left bank of the Natomas East Main Drainage Canal (11.3 miles). On the left bank, the levees extend from Mayhew Drain, downstream to the mouth of the American River (10.8 miles). This study used geotechnical data from previous studies, and in-situ density and laboratory shear strength tests designed to estimate the actual in-situ shear strength of the levee and foundation soils. In addition, levee cross-section surveys performed by the California Department of Water Resource (DWR) in October 1987 were used to determine site specific levee geometry. The DWR surveys included a total of 44 cross-sections from the left levee landside toe to the right levee landside toe. The DWR survey data was plotted

along with estimated flood stage elevations and are presented in Appendix A. The stability of the levees were evaluated for estimated river stages during flows of 115,000 cfs (design flow), 130,000 cfs (February 1986 peak flood flow), and 180,000 cfs. The levee conditions analyzed included stability against landside slope failure during steady seepage conditions and stability against foundation piping (internal erosion).

LEVEE BACKGROUND

The existing left bank levee (10.8 miles) of the American River was brought up to flood control project standards in November 1948 (Ref. 1). This work involved widening of the then existing locally constructed levee from 16th Street to Mayhew Drain (8.8 miles). No work was required of the existing levee downstream of 16th Street. The levee crown was widened to 20 feet with 1V on 3H and 1V on 2H riverside and landside slopes respectively. Following the 1950 flood, two contracts were awarded in 1951 to provide bank protection at three locations. These were just downstream of Hwy 160, downstream of the H Street Bridge, and upstream of the W.P.R.R. crossing. The entire left bank levee is presently maintained by the American River Flood Control District.

In 1955, the right bank levee (3 miles) from the Natomas East Main Drainage Canal upstream to high ground near the California State Exposition (Cal Expo) was brought up to project standards (Ref. 2). This required widening of the levee crown to 20 feet with some minimal increases in levee height. The widening was performed on the riverside portion of the levee with 1V on 2H slopes constructed to meet the existing 1V on 3H slopes. This reach of levee is also presently maintained by the American River Flood Control District.

The right bank levee upstream of Cal Expo, 8.3 miles to a point approximately one mile north of Arden Way, was brought up to project standards in November 1958 (Ref. 3 and 4). Construction involved degrading the then existing 0 to 8-foot high levee and building a new setback levee with a 20-foot wide crown, with 1V on 3H riverside and 1V on 2H landside slopes. Prior to construction, a 6-foot deep, 7-foot wide inspection trench was excavated along the new levee alignment from Cal Expo to the vicinity of Rio Americano High School (R.M. 11), upstream of which the levee height decreases to less than 6 feet. The 4.3 mile levee reach from Cal Expo to Watt Avenue is presently maintained by the California DWR as maintenance area (M.A.) 10. The remaining 4 mile reach upstream of Watt Avenue is maintained by the California DWR as M.A. 11.

GEOTECHNICAL EXPLORATIONS

Based on the as-constructed drawings (Ref. 1), no borings were performed prior to the 1948 levee enlargement on the left bank of the American River. The first explorations along the American River were those conducted in June 1955 by the Corps of Engineers (COE) for the right bank levee enlargement from the Natomas East Main Drainage Canal to Cal Expo. Explorations were also conducted in 1956, 1986, 1987 and 1988. The following paragraphs describe the types of explorations performed. Laboratory testing and soil conditions are discussed in paragraph, LABORATORY TEST RESULTS AND FIELD DENSITY TESTS. The locations of all explorations are shown on Figure 1.

1955

Prior to the 1955 levee enlargement on the right bank from the Natomas East Main Drainage Canal to Cal Expo, five exploratory auger borings (2B-1 to 2B-5) were drilled to determine the suitability of adjacent riverside berm borrow material to be used in the levee widening. No laboratory testing was performed on samples from these borings.

June 1956

In June 1956, a total of 18 borings (2B-1 to 2B-7, 2F-1A to 2F-4, 2F-6, 2F-7, 2F-9, 2F-10, 7F-1 and 5F-8) were drilled along or just adjacent to the selected levee alignment from Cal Expo to high ground about one mile north of Arden Way. Except for borings 2F-1A and 5F-8, which were drilled to determine the foundation conditions for two project pumping facilities associated with the project, the borings were drilled for the purpose of determining levee foundation conditions and to provide additional information on the suitability of adjacent riverside borrow material for the new levee. Except for borings 5F-8 and 7F-1 (6-inch Failing Barrel Sampler), and 2F-1A (24-inch bucket auger), the borings were drilled with 8-inch hand and power augers. Laboratory tests were performed on selected samples (See 1956 laboratory report, Appendix B).

June 1986

In June 1986, four 6-inch auger borings (F-1A to F-4A) with standard penetration tests (SPT) were drilled in the right bank levee of the American River just upstream of Hwy 160. These borings were drilled by the A/E as part of the PL84-99 levee investigation (Ref. 8) where a 1400-foot crack developed along the riverside edge of the levee crown during the February 1986 flood. The borings were drilled from the riverside edge of the

levee crown to depths ranging from 30.0 to 39.0 feet. Samples were collected from each boring for primary laboratory testing (Appendix C).

April 1987

Between April and May 1987, 27 auger borings (DH-1 to DH-12 and DH-21 to DH-31) with SPT data collection, were drilled on both the left and right bank levees of the American River. These explorations were also performed by the A/E as part of the Sacramento River Flood Control System Evaluation. Borings designated with an "A" (example DH-4A) were drilled at the levee landside toe, except for DH-12A, which was drilled at the riverside toe. The borings without the "A" designation were drilled through the levee crown. Laboratory testing of these samples included both soils classification and strength testing. The laboratory test report is included as Appendix D.

October 1987

In October 1987, a field investigation program was undertaken for the purpose of determining the in-situ density and shear strength properties of the levee and foundation soils. Eleven trenches, 4F-87-1A to 4F-87-6B, were excavated in the levee slope and foundation. The trenches designated with an "A" were excavated in the levee slope and the trenches designated with a "B" were excavated in the adjacent levee foundation. Field sand cone density tests were performed and sack samples were collected for laboratory shear strength and permeability testing. The laboratory test results are summarized in the April 1988 laboratory test report, Appendix E. In-situ density test results are summarized in Figure 3.

April 1988

In April 1988 two exploratory trenches, 4F-88-1 and 4F-88-2, were excavated in the right bank levee crown just upstream of Highway 160. This investigation was performed for the purpose of investigating remnant signs of the February 1986 crack in the levee crown and determining the strength characteristics of the levee soils in the area where that 1400-foot crack developed during the flood. The trenches were excavated to depths of 7 and 6 feet respectively. One representative 6-inch diameter, undisturbed, 8-inch long tube sample was collected from each trench for laboratory shear strength testing. The laboratory report summarizing the test results is included as Appendix F. The following paragraphs discuss the results of the laboratory and field testing in this test program as well as those from the previous studies.

LEVEE AND FOUNDATION SOIL CONDITIONS

The results of gradation analyses and Atterberg limits tests are presented in Figure 2. The levee and foundation soils on the right and left banks of the American River are predominately loose to firm silty sand (SM) and sandy silt (ML). The only notable exception is some occurrence of clay on the right bank levee and foundation between Hwy 160 and Hwy 80, at exploration locations F-1A, DH-12, and 4F-88-2 where the soils are predominantly firm sandy clay (CL) and clayey sand (SC-SM). In addition, on the right bank, between the Natomas East Main Drainage Canal (NEMDC) and Hurley Way, the soils are generally much finer grained than upstream of Hurley Way. The fines content (minus 200 sieve size) of the levee and foundation soils in this reach range from about 28 to 85 percent and averages 73 percent, while upstream of Hurley Way the fines content ranges from about 8 to 62 percent and averages 40 percent. Except for the levee soils being generally denser than the underlying foundation soils, there is no appreciable difference in the levee and foundation soils on the right bank levee. On the left bank, the levee and foundation are significantly different, except at borings DH-23 and DH-26 (silty sand-SM), where no appreciable difference was detected. The levee materials on the left bank are relatively clean, containing from about 3 to 50 percent and averaging 11 percent fines. The foundation soils, typically deeper than about 0 to 5 feet beneath the left bank levee-foundation contact are generally much finer and have a fines content ranging from 50 to 98 percent and averaging 59 percent.

Two composite samples, composites A and B, were fabricated from samples collected from trenches 1A through 6B. Composite A, with 31 percent fines, was fabricated from samples from trenches 1A, 1B, 2A, 2B, 3A, 3B, 6A and 6B. Composite A is representative of the silty sand levee and foundation soils found throughout most of the levee system. The finer grained levee soils typically between the NEMDC and Hurley Way on the right bank and most of the left bank foundation are represented by composite B (77 percent fines) which was fabricated from samples from trenches 4A, 4B and 5B. Tests performed on composites A and B are discussed in the following paragraphs.

LABORATORY TEST RESULTS AND FIELD DENSITY TESTS

A conservative estimate of shear strength based on SPT blow count data was used in the initial A/E investigation. However, the soil shear strength values selected for that study were considered too conservative. Therefore, this study used remolded laboratory samples from the levee and foundation soils in order to obtain a better estimate of the in-situ shear

strengths. The following paragraphs describe the laboratory and field test results.

Density Tests

In-situ field densities determined using the sand cone method at trenches 1A through 6B are summarized in Figure 3. In addition, undisturbed densities were determined in 1956 at borings 5F-8 and 2F-2 and are also included in Figure 3. Although a fairly large scatter exists, some correlation between density and percentage of fines is apparent. In general, an increase in density occurs with a decrease in fines content. The average density of the foundation samples is 84.5 pounds per cubic foot (pcf) and the average density of the levee samples is 94 pcf.

Standard compaction tests were also performed on both composites A and B in order to approximate the relative compaction of the levee and foundation soils. Composite A and boring 2B-2 (1956) show good correlation for the silty sand materials. Based on the 110 pcf (standard compaction) maximum dry density of composite A and boring 2B-2, the average relative compaction of the foundation and levee soils are 77 and 85 percent respectively. The relative compaction of the levee fill may be conservative since the in-situ densities were performed on the outer portions of the levee where compaction is generally lower than the interior portions of the levee fill.

Shear Strength Tests

In order to correlate in-situ shear strengths with laboratory shear test results, the composites were remolded to various densities prior to performing shear tests (Figure 4). Composite A was remolded to 80, 95, and 110 pcf, while composite B was remolded to 80, 90, and 100 pcf. Ten direct shear tests of silty sand, sandy silt and one lean clay sample were performed. Direct shear strength for composite A and B samples ranged from 0=35.5 to 41.0 degrees, with slightly lower results from two of the three 1956 test samples. Five consolidated-undrained shear tests were also performed. Shear strengths for these tests ranged from 0=5 degrees with a cohesion (c)=40 psf to 0=15 degrees with c=400 psf.

Permeability Tests

Permeability tests were performed on composites A and B (Figure 5). The vertical permeability (K_v) of composite A ranged from approximately 0.1 to 3 feet per day, depending on the remolded density. Composite B permeability (K_v) ranged from 0.01 to 0.07 feet per day. Since there is a wide variation in soil gradations for the levee and foundation materials, these values only provide

an indication of the order of magnitude of the permeability of the levee and foundation soils. However, the permeability of composite A is considered representative of the levee and foundation soils on the right bank upstream of Hurley Way. The levee and foundation soils on the right bank downstream of Hurley Way as well as the foundation soils on the left bank are more represented by Composite B. Explorations on the left bank levees indicated some pockets or stratified layers of fine to medium sand with 3 to 10 percent fines. It is estimated that the permeability of these isolated materials is on the order of 100 feet per day. However, since no seepage was reported through the left bank levees, it is possible that these cleaner layers are not continuous.

SELECTED VALUES FOR ANALYSES

Shear strength values selected for slope stability analyses are shown on Figure 4. An effective strength of 0=34 degrees was considered representative for the levee soils. Since the density of the foundation soils are typically about 10 pcf less than the levee fill, a slightly lower foundation strength of 0=31 degrees was selected. A consolidated-undrained strength value of 0=12 degrees, and c=360 psf was selected for the levee soils, while a shear strength of 0=12 degrees and a cohesion of 180 psf was selected for the foundation soils. Also indicated in Figure 4 are the composite shear strength envelopes selected for the stability analyses. These strength envelopes reflect the composite shear strength criteria as required for steady state seepage conditions (EM-1110-2-1903) and were used in the stability analyses shown on Figure 6.

The only permeability tests available were those performed on composite A (silty sand) and composite B (sandy silt). The results of these tests were discussed in paragraph 6-c. In order to account for potential continuous clean layers of fine sand on the right bank levee and somewhat cleaner sands on the left bank, conservative values of K_v=30 and K_h=120 feet per day are considered representative of the levee and foundation soils on the right bank, upstream of Hurley Way and representative of the entire left bank levee. The permeability of composite B, typifying the generally finer grained, lower permeability soils downstream of Hurley Way and most of the foundation soils of the left bank, ranged from approximately 0.01 to 0.07 feet per day. For the purpose of estimating seepage exit gradients and piping stability, the foundation soils deeper than five feet are conservatively assumed to be impermeable.

ANALYSES

There are approximately 22 miles of levee protecting the land north and south of the American River. There are variations in: levee height; hydraulic head differences between river stages and adjacent land surface; freeboard; and levee and foundation soil characteristics. Therefore, in order to make a practical assessment of levee stability, conservative, yet reasonable values of levee and foundation shear strength and permeability were selected. In addition, site specific levee geometry as determined from the 44 California DWR, cross-section surveys mentioned previously and flood stage profile data were used in estimating freeboard, hydraulic head and levee seepage exit heights for determining levee slope stability and piping potential (Appendix A). The flood stage elevations for the February 1986 flood (130,000 cfs), were obtained from high water marks and the stream gauge on the H Street Bridge. The flood stage elevations for 115,000 cfs and 180,000 cfs were obtained from the preliminary water surface elevations computed for the 1988 Federal Emergency Management Agency (FEMA) flood insurance study of the City and County of Sacramento. For a levee section to be considered stable, three criteria were established. These criteria included: 1) meeting a minimum freeboard of 3 feet; 2) having a factor of safety for slope stability of 1.4 and; 3) having a factor of safety against piping of 3.0. Three feet of freeboard was used in this analysis only as a measure of initial freeboard safety. The analysis in this chapter is primarily a structural stability analysis and criteria 2 and 3 are most important. Actual design freeboard for all levee measures evaluated is discussed in Chapter 1 of Appendix N. A typical levee section (20-foot crown, 1V on 3H waterside and 1V on 2H landside slopes) as determined from as-constructed drawings, was selected in order to determine the critical seepage exit point for slope stability. Flow nets were also developed for a typical section for the purpose of estimating the critical head difference (with regard to piping potential) between the river stage and the adjacent land surface. The following two paragraphs describe in more detail the methods used and the results achieved for slope stability and piping stability.

Slope Stability

The results of the slope stability analyses (Modified Swedish Method) are presented in Figure 6. The critical condition for slope stability of the levees occurs under a steady seepage condition. For the purpose of this analysis, this condition was assumed for three different flood stages with the corresponding freeboard based on a relatively high levee section (section 38-right bank), just downstream of Watt Avenue. Using the

estimated permeability value of k_v=30 feet per day and a selected conservative in-situ void ratio (e) of 1.0, it is estimated that a steady seepage condition could develop in about 4 days for a flow of 115,000 cfs, 3 days for a flow of 130,000 cfs, and only 1 or 2 days for a flow of 180,000 cfs. A conservative assumption used in this analysis (Huang, Ref. 7) assumes an impervious foundation. For most of the left bank levee, this assumption is fairly accurate. On the right bank, where no significant difference exists between the levee and foundation soils, the assumption is on the conservative side.

Where seepage water saturates the landside levee slope, the stability analyses results in very shallow slope failures. The potential for slope failure or sloughing of the upper one foot of the levee slope was ignored for two reasons. First, the analyses discounts apparent cohesion at very low stresses. In fact, vegetation (grass roots) in the upper 6 to 12 inches of the levee slope does provide a stabilizing effect due to root reinforcement. Second, if shallow sloughing does develop in this zone, the rigorous levee inspection practices presently used during flood releases would insure that emergency procedures are used to stabilize the levee. Therefore, only failure arcs below this 1-foot zone were considered critical. A flow of 130,000 cfs for the typical section resulted in a seepage exit height above the levee toe that caused a borderline stability condition. At this flow, the seepage water exit point was calculated to be 0.63 feet above the landside levee toe. A factor of safety of less than 1.4 (minimum required by EM1110-2-1901) develops below the upper one foot surface at a flow of 130,000 cfs. In addition, a failure surface with a factor of safety of 1.34 develops three feet below the ground surface. The steady seepage exit point above the landside levee toe was determined by the L. Casagrande method (Ref. 5). Table 1 includes this seepage exit height as well as the parameters used in the calculations for each of the 44 DWR cross-sections. Although at some locations the levee landside slopes surveyed are flatter than 1V on 2H, the 1V on 2H slope was conservatively used throughout the analyses. Therefore, levee sections with a calculated seepage exit height, based on levee geometry and head differential, greater than a value of 0.60 feet are considered potentially unstable. Seepage exit heights for each levee section are also summarized on Table 2. The results for a flow of 115,000 cfs show that the maximum exit height is 0.45 feet. It is noted that several sections on the right bank, downstream of Cal Expo (cross-section 17), have significantly greater head differentials between the river stage and the adjacent land. This reach is not considered critical because of the relatively impervious nature of the levee and foundation soils in this reach. Permeability of the levee and foundation soils in this reach are typically less than 0.10 feet per day. It is estimated that steady seepage conditions would

take one to two months to develop in this area. This is much longer than the probable flood durations of less than one week.

Piping Stability

In order to evaluate the potential for piping, flow nets were developed for various hydraulic head differences between river flood stage and the adjacent land surface (Figure 7). For the purpose of flow net construction, horizontal permeability was assumed to be four times the vertical permeability. As discussed previously, the difference in levee and foundation soils on the right bank is negligible. The left bank foundation soils, typically beginning at a depth of approximately five feet are much finer grained and are relatively impervious compared to the silty sand and sand levee soils. Therefore, the typical levee section, with an assumed impermeable foundation at a depth of 5 feet was selected for the purpose of developing flow nets and determining seepage exit gradients and the factor of safety against piping. Seepage exit gradients were calculated for hydraulic head differences of 5, 6 and 7 feet. A minimum required factor of safety against piping of 3.0 was selected. In order to maintain this factor of safety, the maximum hydraulic head differential between the water surface and landside toe must be less than 6.0 feet (Figure 7). This value was used in this study to identify locations that may be susceptible to piping. Values that exceed 6.0 feet are underlined on Table 2. The results on Table 2 indicate that none of 44 cross-section analyzed exceed this value for the design flood of 115,000 cfs. At a flow of 130,000 cfs, five levee sections exceed the 6.0-foot criteria and at 180,000 cfs, 6.0 feet is exceeded in several areas. It is noted that downstream of about Hurley Way (cross-section 22) on the right bank, the head differential is much greater than 6.0 feet in several locations. As previously discussed in paragraph 7-a, the permeability of the levee and foundation soils on the right bank downstream of Hurley Way are generally sandy silt, silt and clay soils of very low permeability. It is estimated that the duration of flood water on the levee in this area must exceed at least one month before steady seepage conditions could develop. Therefore, the greater head differentials on the right bank downstream of Hurley Way are not considered significant.

CONCLUSIONS

Three criteria were used in this study to evaluate the stability of 44 levee cross-sections (See Table 2). For the levee sections to be considered stable, it was determined that all three criteria should be meet. These criteria include: 1) a minimum of 3 feet of freeboard; 2) an estimated steady seepage

water exit height above the landside levee toe of no more than 0.60 feet; and 3) a hydraulic head difference between flood stage and the adjacent landside levee toe of no more than 6.0 feet. Actual structural stability is represented by criteria 2 and 3 only. Freeboard was evaluated as an indication of freeboard safety. Actual design freeboard required is discussed in Chapter 1 of Appendix N. Criteria 2 and 3 were established based on stability analyses and flow net analyses (piping stability) on typical levee sections. Flood stages of 115,000 cfs (design flood), 130,000 cfs (peak February 1986 flood stage), and 180,000 cfs (100-year flood) were evaluated. This study concludes the American River levees are structurally stable for flows not exceeding the design flow of 115,000 cfs. At a flow of 130,000 cfs, five locations exceed at least one criteria. Although no levee instability or piping developed on the American River levees during the 1986 flood, the analyses indicate that an extended flow of 130,000 cfs or greater would likely lead to landside levee sloughing and/or piping in some locations. The maximum allowable duration at which no levee damage would occur for a flow of 130,000 cfs is difficult to predict because of unknowns such as: 1) duration of lower flood stages such as 115,000 cfs and 2) variations in levee soils and permeability as well as the potential for continuous stratified deposits. However, using the selected permeability values, the length of time required to develop a steady seepage condition at 130,000 cfs is estimated to be roughly three days. Because of unknowns and potential anomalies, one day should be considered the critical duration for a flow of 130,000 cfs. At 180,000 cfs, the minimum criteria is exceeded in several areas and potential for levee damage or failure is very high.

RECOMMENDATIONS

If flows in the American River do not exceed the design flow of 115,000 cfs, remedial repairs are not considered necessary. However, in view of past experience, flood releases to the American River may have to be increased above the design capacity during extreme flood conditions. Therefore, assuming no upstream modifications are made to the present flood control system, modifications are recommended over portions of the levee system to accommodate a flood release of at least 130,000 cfs. Remedial repairs would likely include those areas near sections underlined on Table 2. Table 2 should not be interpreted as providing the exact limits of repair. It does, however, provide an indication of the magnitude of remedial repair needed. Minimal portions of the levee system need some modifications to be considered stable at a flow of 130,000 cfs. It is estimated that the remedial repairs would involve roughly 4500 feet on the left bank and 2500 feet on the right bank. More precise

determinations of the limits for remedial repairs could be made with additional survey work in the areas identified on Table 2. The types of repairs that would likely be considered are drained buttresses where space is available and slurry cutoff walls where space is limited (See Figure 8). It should also be pointed out that ongoing erosion occurs on portions of the levee riverside berms during high river flows. In some instances complete loss of the riverside berm has caused portions of the levee to slough. This was the case during the February 1986 flood when extensive damage occurred to the left bank levee immediately upstream of Highway 80. 1986 flows exceeded 130,000 cfs for 20-30 hours.

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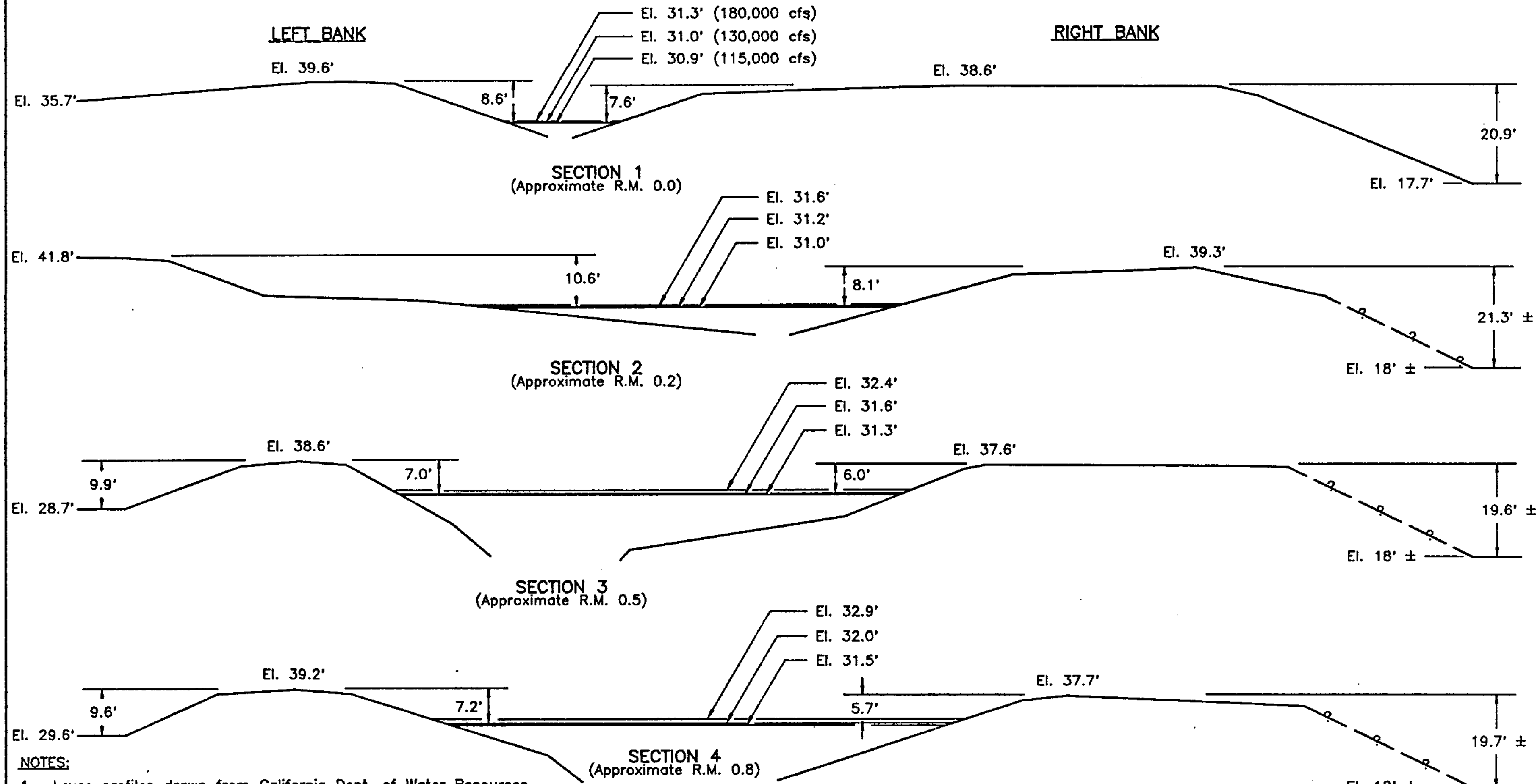
APPENDIX M-2-A

LEVEE AND FLOOD PROFILES



NOTES:

1. Levee profiles drawn from California Dept. of Water Resources October 1987 Cross-Section Surveys of the American River.
2. River stage profiles indicate flood releases from Nimbus Dam of 115,000 cfs, 130,000 cfs, and 180,000 cfs. These stages are based on surveyed high water marks from the February 1986 flood (130,000 cfs peak) and from preliminary water surface elevations computed for the 1988 FEMA Flood Insurance Study of the City and County of Sacramento by the Sacramento District, U.S. Army Corps of Engineers.
3. Dimensions drawn to water surface approximate the minimum freeboard occuring during the February 1986 flood.
4. Sections are drawn looking downstream.
5. Cross-sections depict only the left and right bank levee sections, and not the entire channel width.

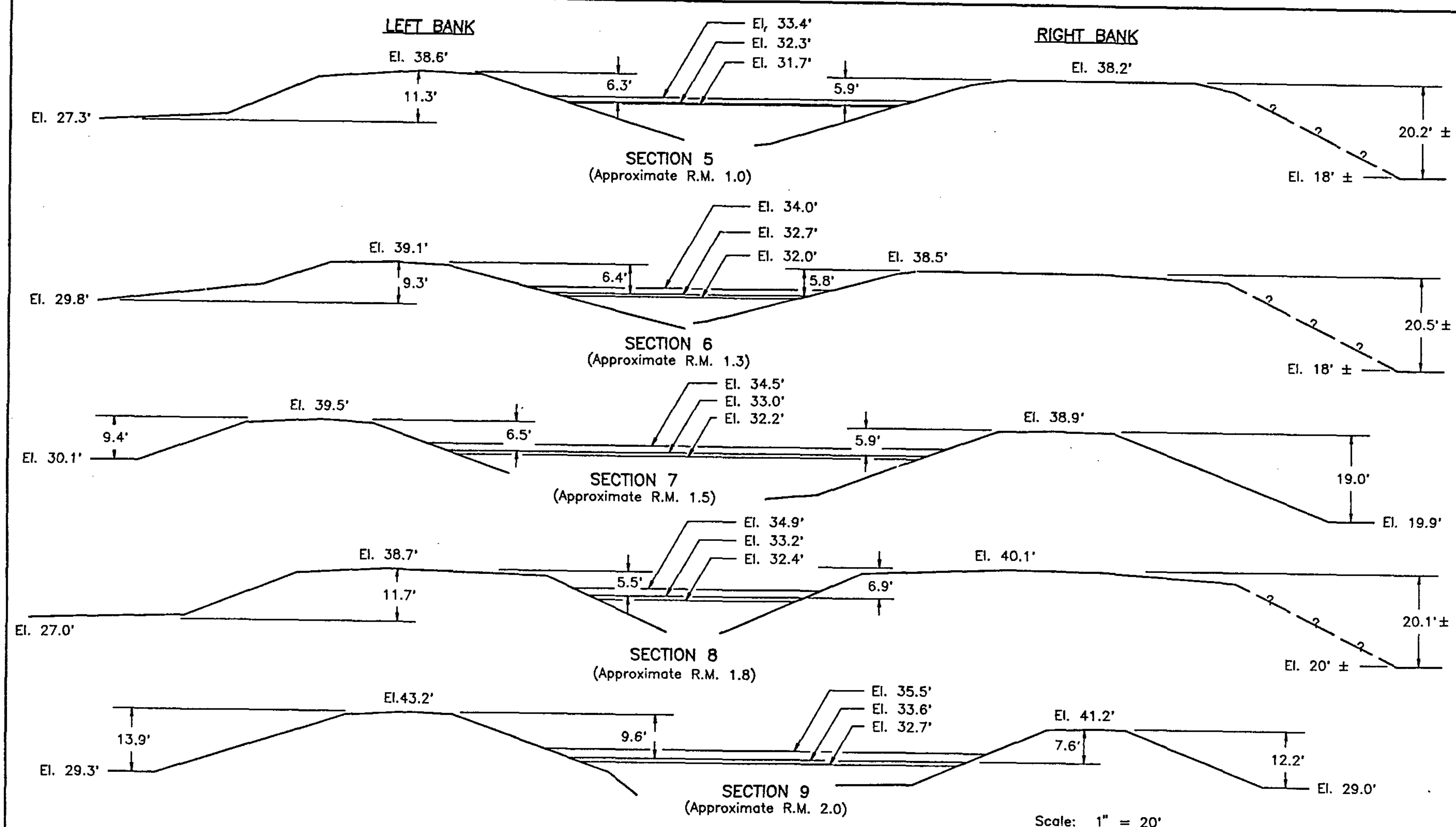
Scale: 1" = 20'

SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

LEVEE AND FLOOD PROFILES

SECTIONS 1 through 4

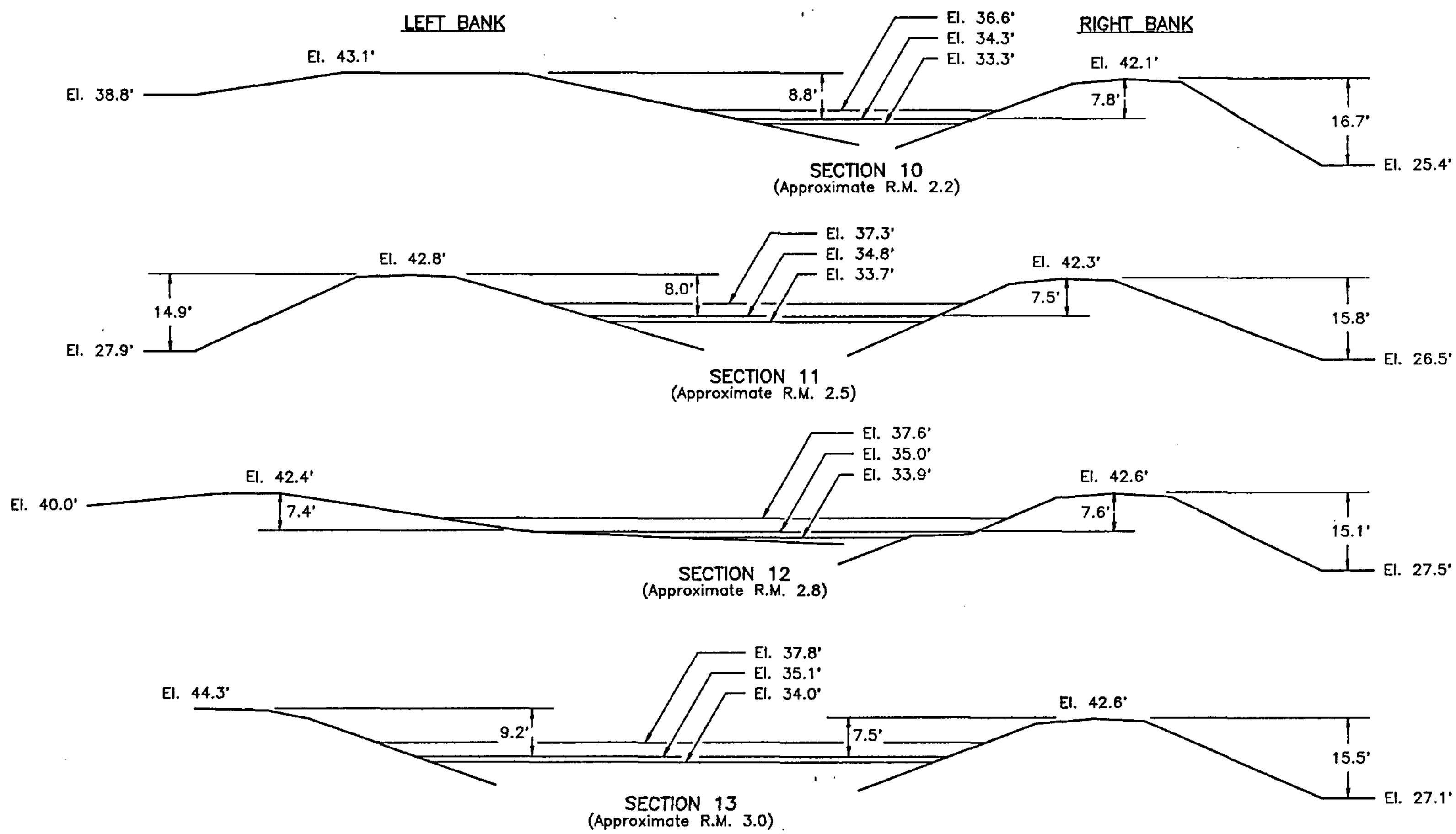


SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

LEVEE AND FLOOD PROFILES

SECTIONS 5 through 9



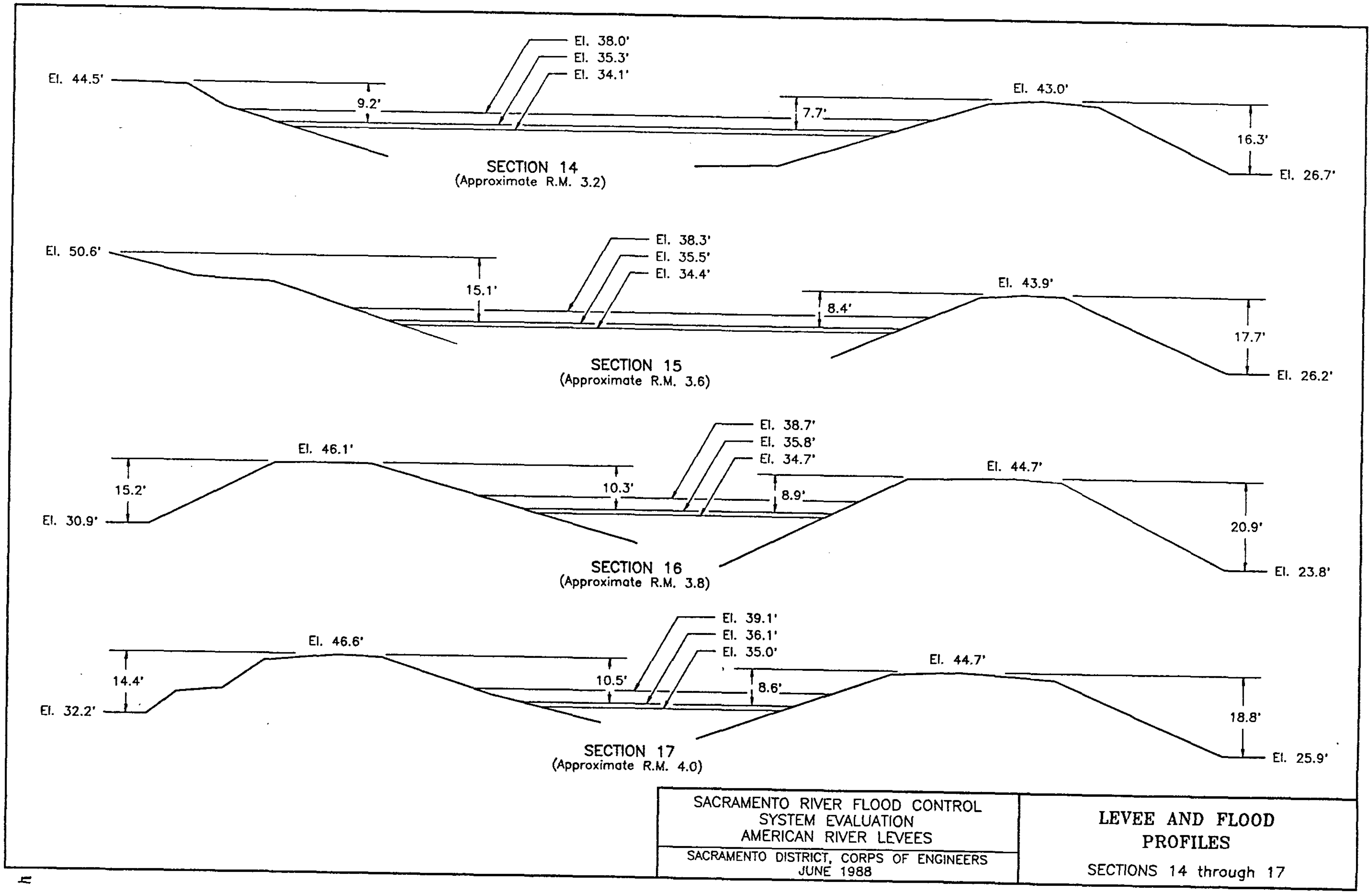
Scale: 1" = 20'

SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

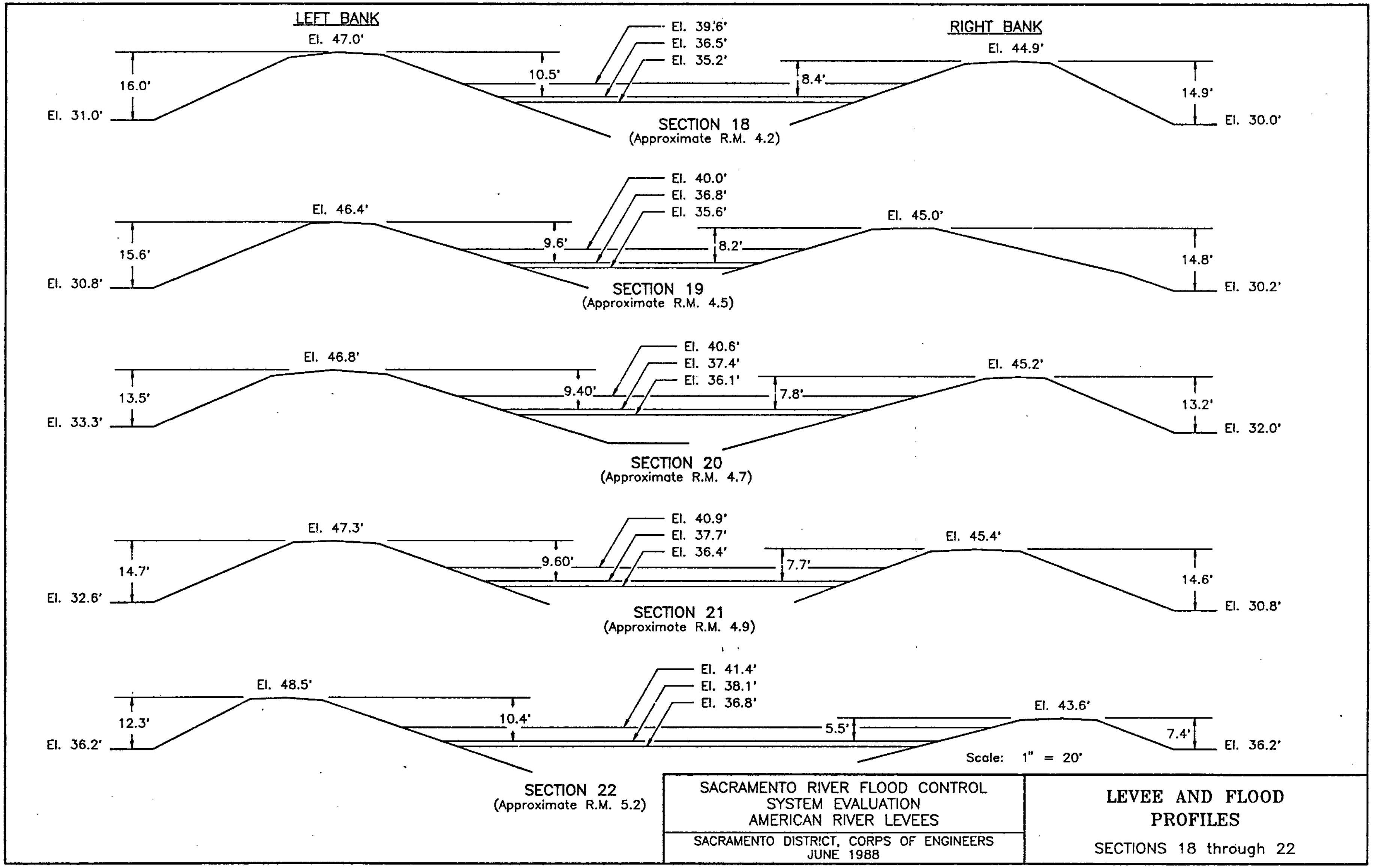
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

LEVEE AND FLOOD PROFILES

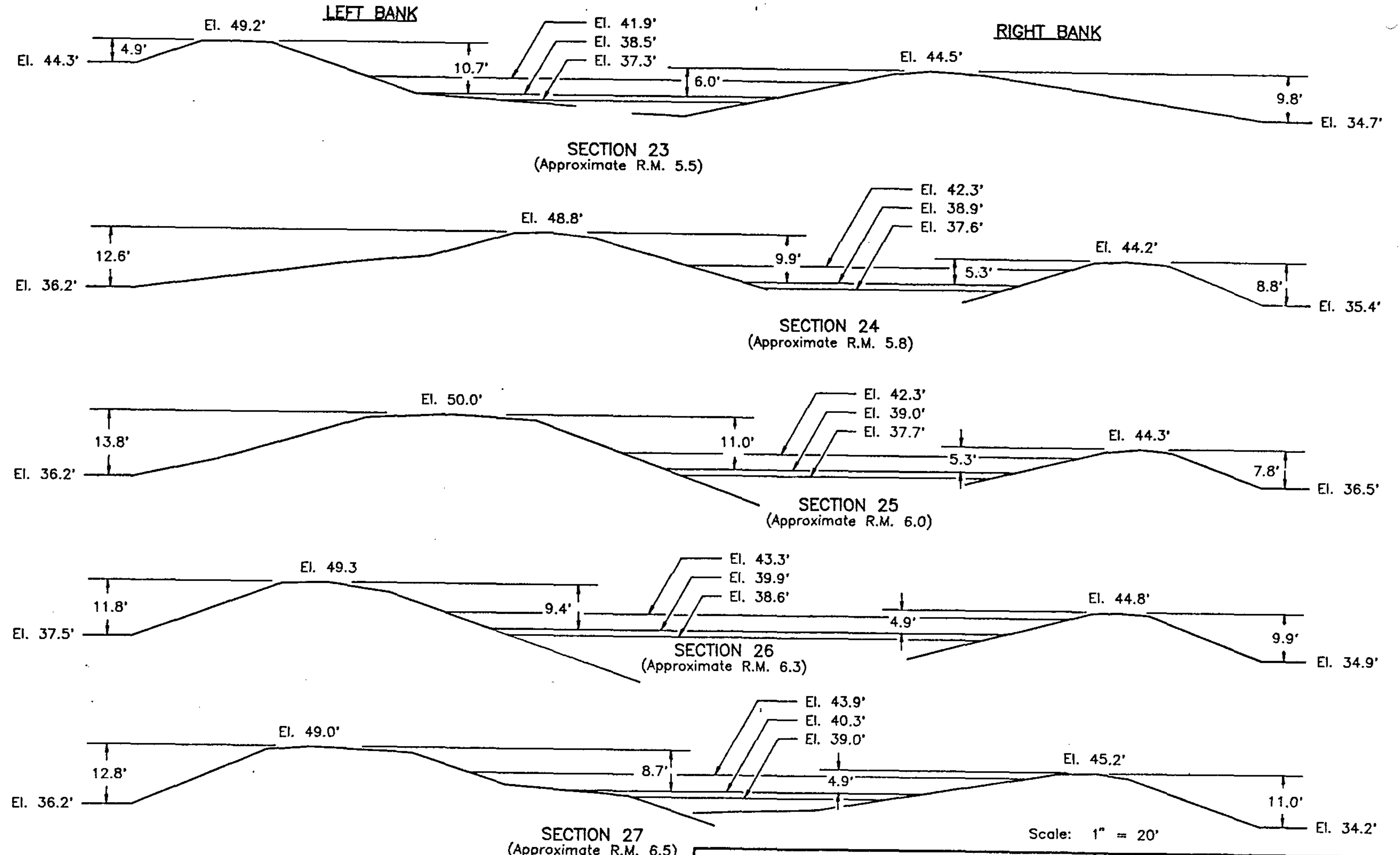
SECTIONS 10 through 13


El. 44.5'
El. 38.0'
El. 35.3'
El. 34.1'
9.2'
7.7'
El. 43.0'
16.3'
El. 26.7'
SECTION 14
(Approximate R.M. 3.2)
El. 50.6'
El. 38.3'
El. 35.5'
El. 34.4'
15.1'
8.4'
El. 43.9'
17.7'
El. 26.2'
SECTION 15
(Approximate R.M. 3.6)
El. 46.1'
El. 38.7'
El. 35.8'
El. 34.7'
15.2'
El. 30.9'
10.3'
8.9'
El. 44.7'
20.9'
El. 23.8'
SECTION 16
(Approximate R.M. 3.8)
El. 46.6'
El. 39.1'
El. 36.1'
El. 35.0'
14.4'
El. 32.2'
10.5'
8.6'
El. 44.7'
18.8'
El. 25.9'
SECTION 17
(Approximate R.M. 4.0)
SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988
LEVEE AND FLOOD
PROFILES
SECTIONS 14 through 17

4



LEFT BANK
RIGHT BANK
El. 47.0'
El. 31.0'
16.0'
10.5'
El. 39.6'
El. 36.5'
El. 35.2'
8.4'
El. 44.9'
14.9'
El. 30.0'
SECTION 18
(Approximate R.M. 4.2)
El. 46.4'
El. 30.8'
15.6'
9.6'
El. 40.0'
El. 36.8'
El. 35.6'
8.2'
El. 45.0'
14.8'
El. 30.2'
SECTION 19
(Approximate R.M. 4.5)
El. 46.8'
El. 33.3'
13.5'
9.40'
El. 40.6'
El. 37.4'
El. 36.1'
7.8'
El. 45.2'
13.2'
El. 32.0'
SECTION 20
(Approximate R.M. 4.7)
El. 47.3'
El. 32.6'
14.7'
9.60'
El. 40.9'
El. 37.7'
El. 36.4'
7.7'
El. 45.4'
14.6'
El. 30.8'
SECTION 21
(Approximate R.M. 4.9)
El. 48.5'
El. 36.2'
12.3'
10.4'
El. 41.4'
El. 38.1'
El. 36.8'
5.5'
El. 43.6'
7.4'
El. 36.2'
Scale: 1" = 20'
SECTION 22
(Approximate R.M. 5.2)
SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988
LEVEE AND FLOOD
PROFILES
SECTIONS 18 through 22

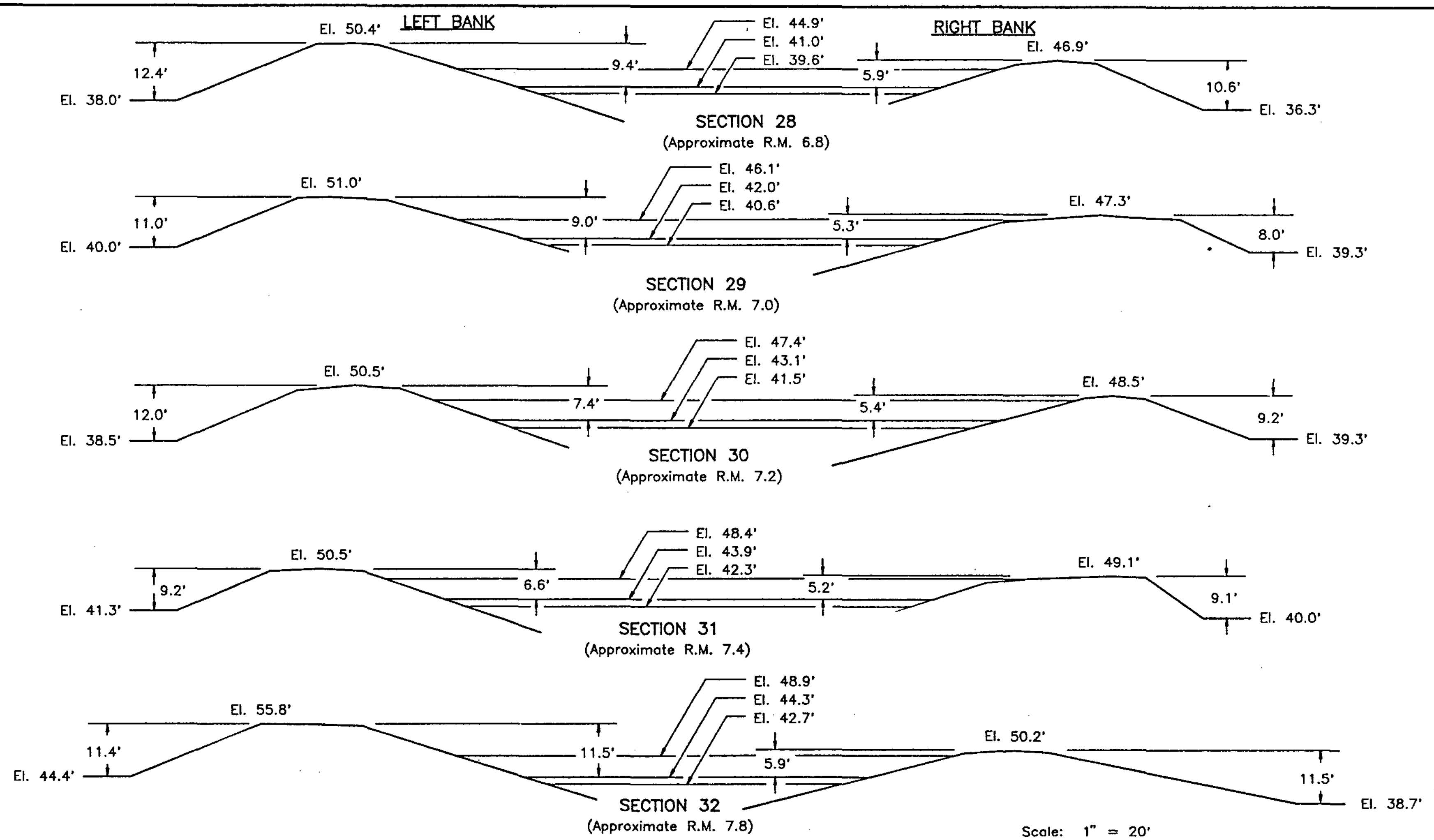


SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

LEVEE AND FLOOD PROFILES

SECTIONS 23 through 27

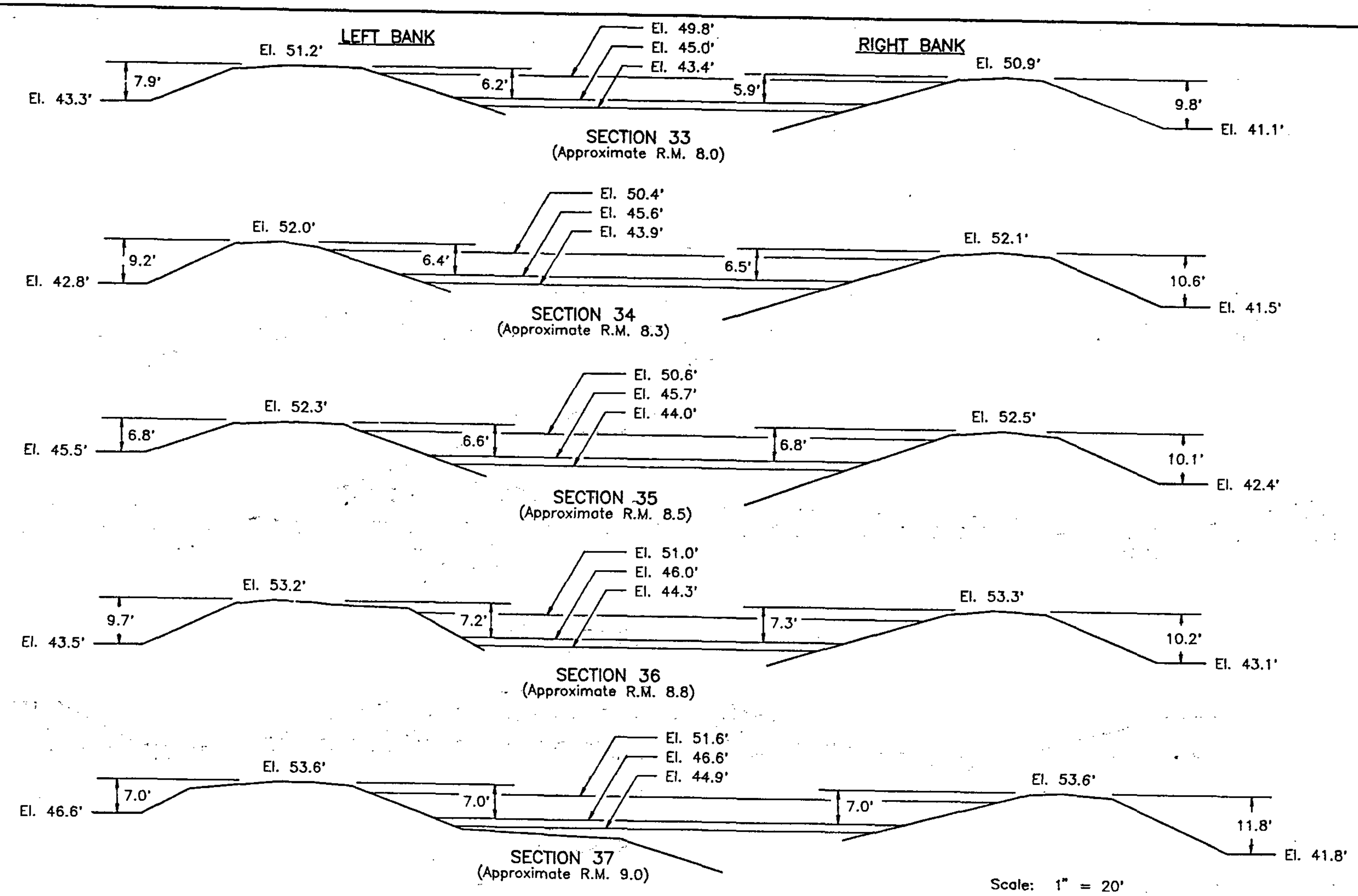


SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

LEVEE AND FLOOD PROFILES

SECTIONS 28 through 32

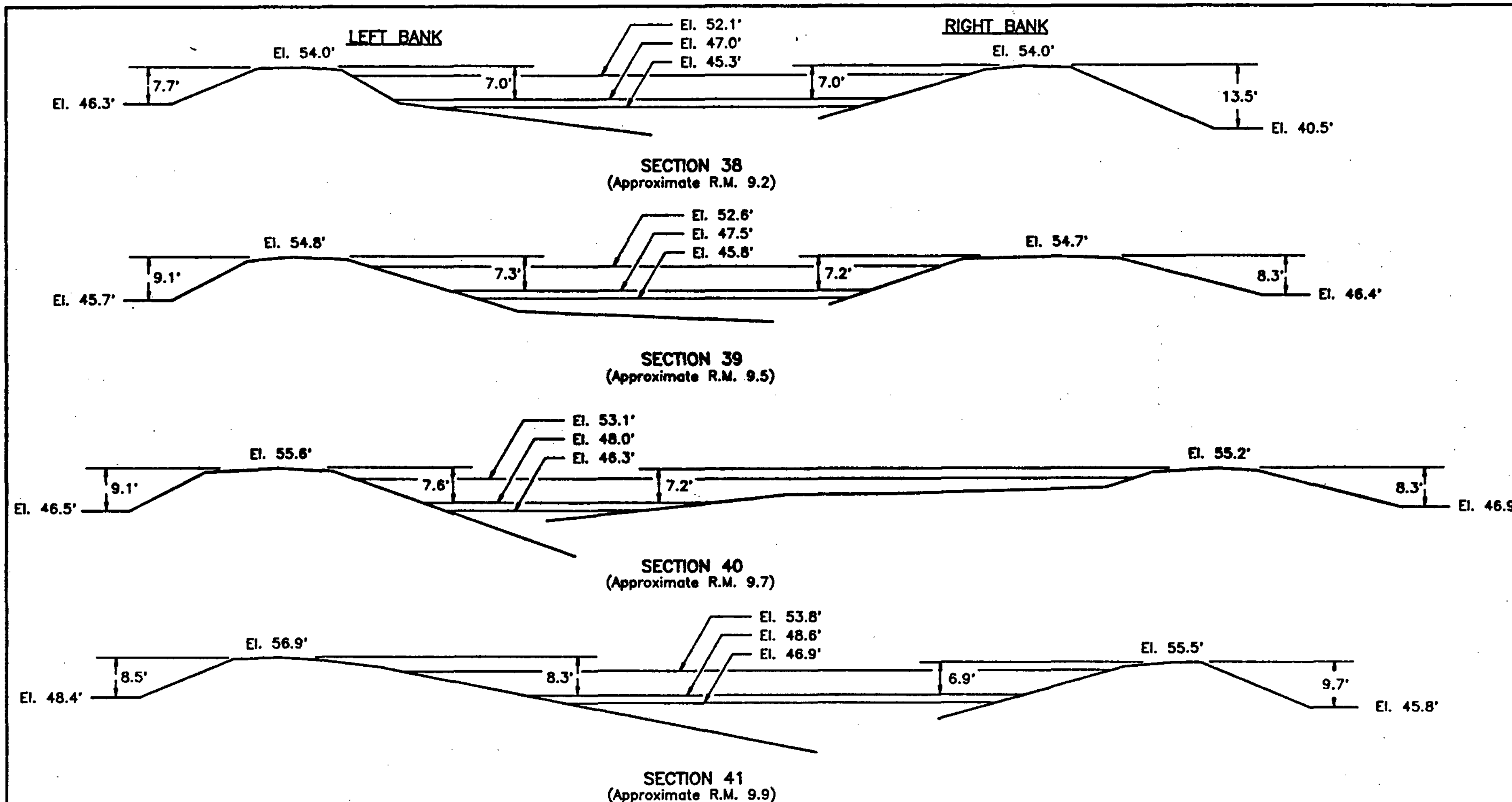


SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

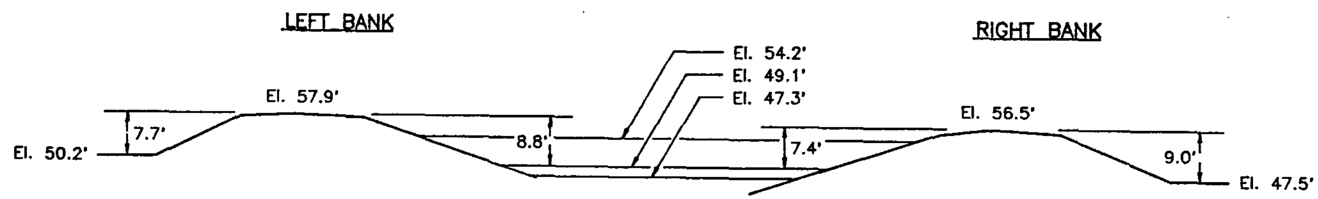
LEVEE AND FLOOD PROFILES

SECTIONS 33 through 37

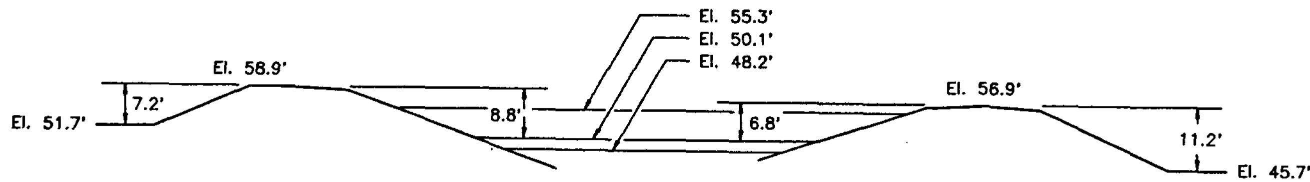


Scale: 1" = 20'

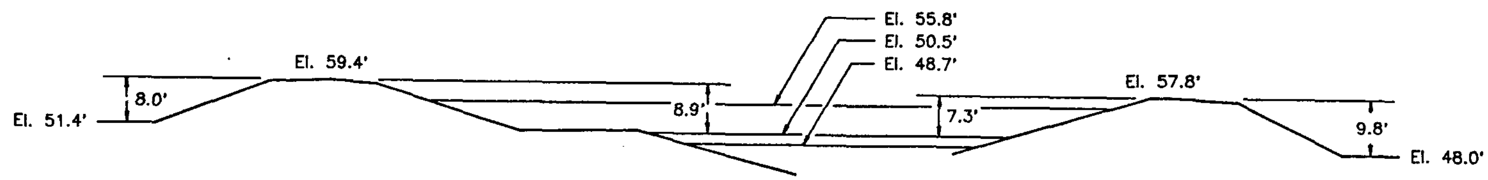
SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION AMERICAN RIVER LEVEES	LEVEE AND FLOOD PROFILES SECTIONS 38 through 41
SACRAMENTO DISTRICT, CORPS OF ENGINEERS JUNE 1988	



SECTION 42
(Approximate R.M. 10.2)



SECTION 43
(Approximate R.M. 10.5)



SECTION 44
(Approximate R.M. 10.7)

Scale: 1" = 20'

SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION AMERICAN RIVER LEVEES	**LEVEE AND FLOOD PROFILES** SECTIONS 42 through 44
SACRAMENTO DISTRICT, CORPS OF ENGINEERS JUNE 1988	

APPENDIX M-2-B

REPORT OF SOIL TESTS - AMERICAN RIVER, OCTOBER 1956

REPORT
OF
SOIL TESTS

AMERICAN RIVER RIGHT BANK LEVEE

FAIRGROUNDS TO CARMICHAEL BLUFFS

October 1956

1. Authorization. Tests reported herein were verbally requested by Mr. D. Doble of the Sacramento District, 13 July 1956, and confirmed by letter dated 18 July 1956, file SPKGD-C 600.95, subject: "Request for Tests - American River Right Bank Levee - Fairgrounds to Carmichael Bluffs."

2. Purpose. The purpose of the tests was to provide data to aid in the design of a pumping plant foundation and the design of a levee cross section.

3. Samples. The samples from this project were received 22 and 28 July 1956, and represent either disturbed or undisturbed material obtained from seven test holes. Only a portion of the samples were tested in this program; samples not tested were returned to the Sacramento District. Samples for which tests were made are shown on "Test Data Summary" plates 1 and 2.

4. Testing Program. The testing program was essentially that as outlined in the letter of request and consisted of the following:

a. Visual classifications, with sufficient mechanical analyses and Atterberg limits determinations to verify the visual classification on all samples from hole 5F-8. In addition, two field density and moisture determinations were obtained from each of the samples from this hole.

b. Laboratory classification and direct shear tests on the undisturbed sample from hole 2F-2 (Dist. No. C-1418-56).

c. Laboratory classification, compaction and direct shear tests on specimens remolded to 95% Standard AASHO density on samples from hole 2B-2 (Dist. No. C-1433-56) and hole 2B-6 (Dist. No. C-1473-56). It was originally planned that direct shear tests on sample C-1473-56 would be made on specimens remolded to 95% Standard AASHO density as determined for C-1433-56, but since the maximum densities of the two

materials differed by approximately 13%, tests were made on C-1473-56 at 95% maximum density as found for C-1473-56.

5. Test Methods.

a. Mechanical Analysis. The grain-size distribution was found by: (1) washing a representative portion of the entire sample on a No. 200 sieve (since there were no gravel sizes present) after the soil had been oven-dried and then slaked in water over night; and (2) making a dry sieve analysis of the retained portion. Where fifty or more percent of fines was indicated a combined sieve and hydrometer analysis was made in general accordance with ASTM Designation D422-54T.

b. Atterberg Limits. Liquid and plastic limit determinations were made in general accordance with ASTM Designations D423-54T and D424-54T, respectively. The only deviation from the standard procedure was the use of the Casagrande grooving tool in the liquid limit determinations. Where no 1/8" diameter threads could be rolled from the moist soil the material was designated as non-plastic and no further tests were made.

c. Classification. Soil classification was made in accordance with the "Unified Soil Classification System, Appendix A, Volume 2, March 1953."

d. Specific Gravity. Determination of specific gravity was made by the pycnometer bottle method, using a high vacuum to free the soil of adsorbed air.

e. Field Dry Unit Weight and Moisture Content. Determination of unit weight was found by the waxed chunk method, with moisture content being part of the test. The material from sample C-1506-56 (Div. No. 12407-SA) was too cohesionless to obtain a chunk specimen, but a range of density at field moisture was investigated by placing the material as loosely as possible in a volumetric measure and also by vibrating the material by tapping the container until no appreciable consolidation took place.

f. Compaction. Maximum density and optimum moisture were found in general accordance with AASHO Designation T-99-49. Deviation from the procedure was that new material was used for each point of the compaction curve and the moisture was added to the soil approximately eighteen hours before compaction to insure even distribution.

g. Direct Shear. Consolidated-drained direct shear tests were performed on undisturbed specimens from hole 2F-2 and specimens remolded to 95% Standard AASHO density from the disturbed material

from holes 2B-2 and 2B-6. Three companion specimens $3\frac{1}{2}$ inches square and 1-inch in height were consolidated under test normal load, with access to water, at least over night before shearing. Normal loads of 0.5, 1.0 and 1.5 tons per square foot were used. Shearing was accomplished by applying increments of strain at time intervals sufficient to permit drainage and allowing the "stressometer" to relax for almost the entire time interval before recording the registered load. The rate of strain was essentially constant, with the average total test time including time during the night in which no strain increment was applied, shown on the "Direct Shear Test Report" plates. Free drainage of the soil was permitted by porous stones on the top and bottom of the specimen.

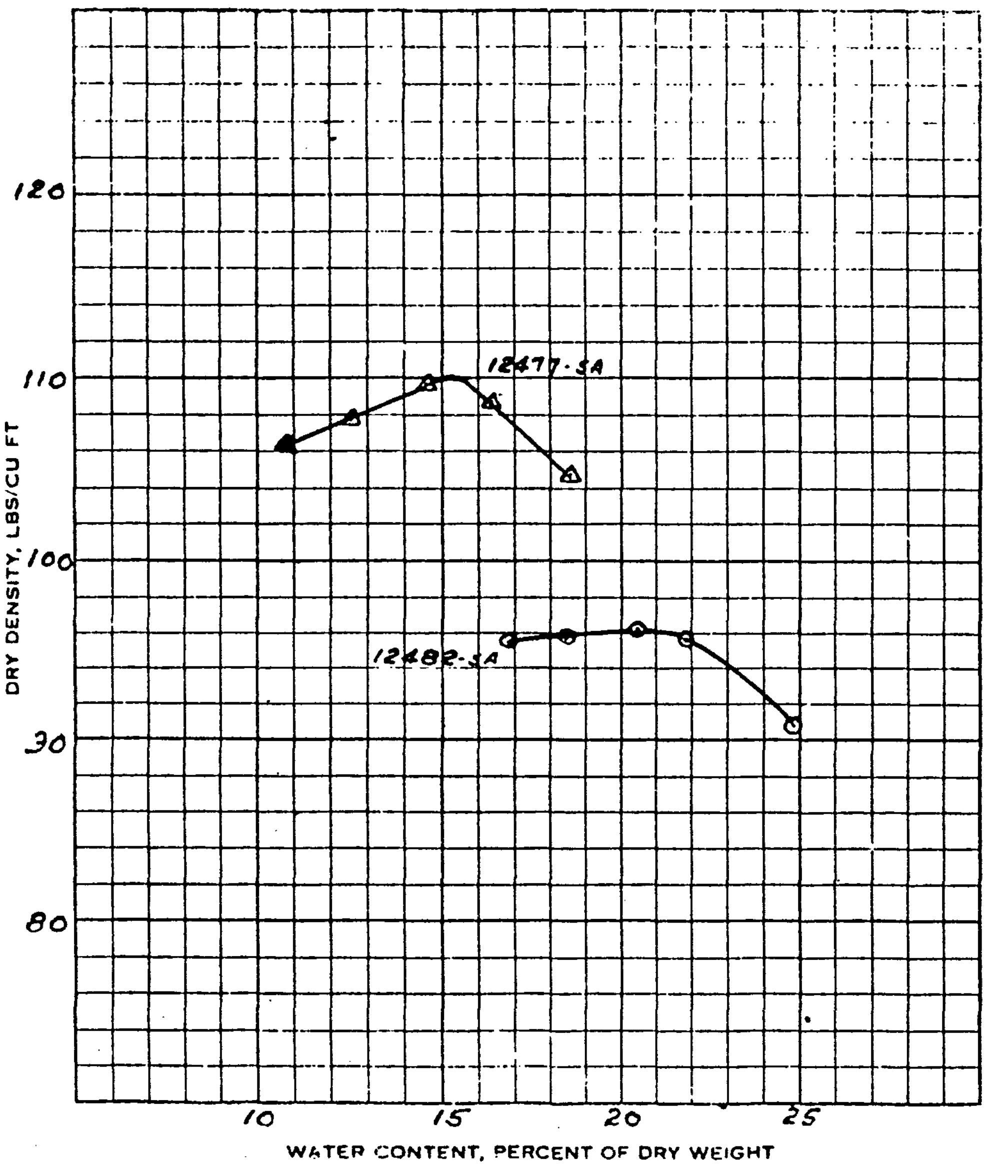
6. Test Results. The results of the tests reported herein are shown on the following plates:

Subject	Plate No.
Test Data Summary	1 - 2
Sample Log	3 - 4
Mechanical Analyses Plot	5 - 6
Compaction Test Report	7
Direct Shear Test Report	8 - 10

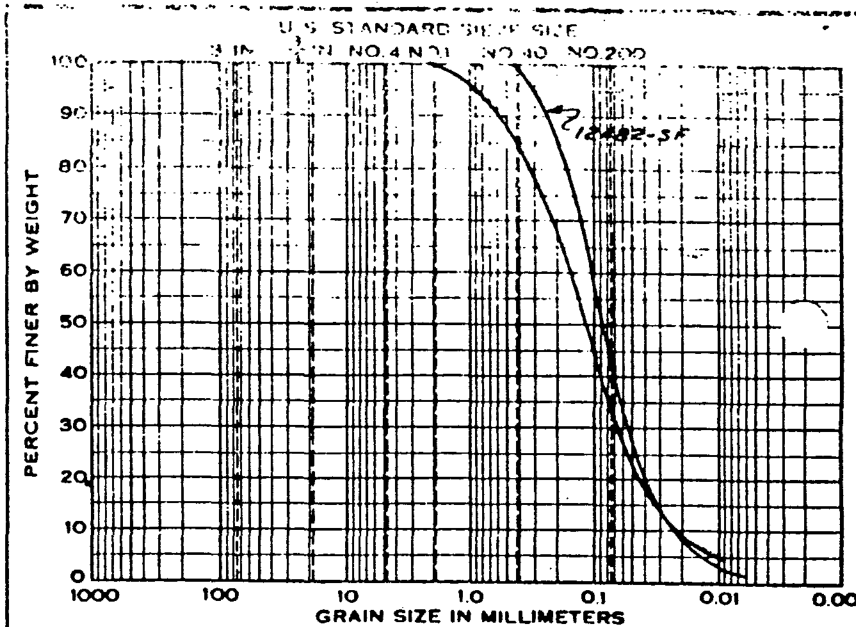
7. Test Observations.

a. General. The materials encountered were, for the most part, silty sands with test hole 5F-8 showing a stratum of very wet sandy silt at a depth of about ten feet. All of the sample densities appeared quite low which might indicate that appreciable settlement would take place in the foundation of the pumping station.

b. Direct Shear. Results of shear tests showed no unusual behavior. Shear values appear a little lower than those usually found for silty sands or sandy silts, however, it is to be noted that the densities of all of the materials were quite low. The lower shear angle found for 12482-SA, in comparison with 12477-SA, is probably due to the lower density of the specimens, although the volume changes as well as stress-strain curves developed during the test cycle indicate that relatively dense conditions were obtained during consolidation and that the densities at start of shear may have been quite comparable.



TEST METHOD :- STANDARD A.A.S.H.O. T99-49



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Elev or Depth	Classification	G	LL	PL
12477-SA	0-6'	SILTY SAND (SM)	2.70	-	NP
12482-SA	0.4'-3'	" " "	2.73	-	NP

Sample No.		12477-SA	12482-SA		
Optimum Water Content	%	15.3	21.0		
Max Dry Density	Lbs/Cu Ft	110.0	96.2		
Optimum Water Content Corr for +	%				
Max Density Corr for +	Lbs/Cu Ft				

Project AMERICAN RIVER LEVEE

Area

Boring No. 2B-2 = 12477-SF, 2B-6 = 12482-SF

Sample No.

Elev or Depth

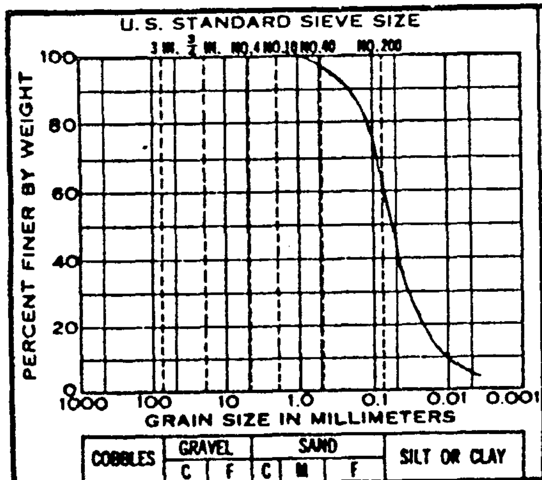
Date

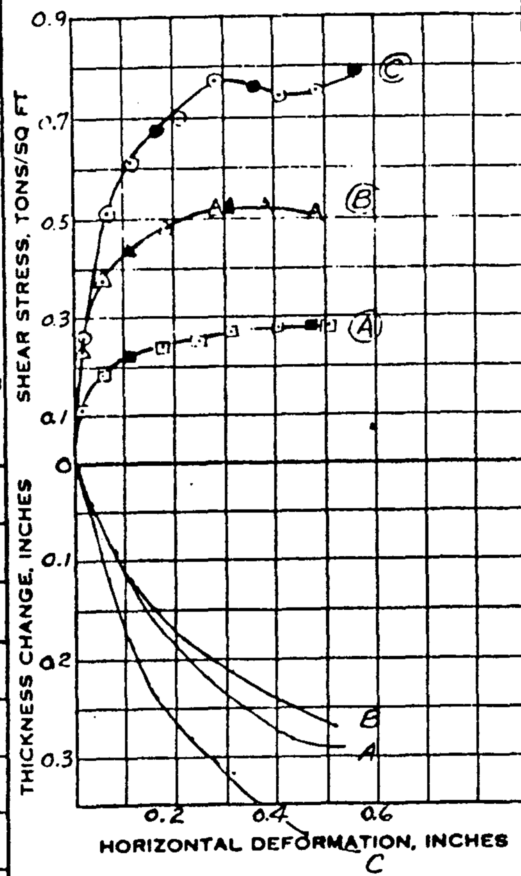
COMPACTION TEST REPORT

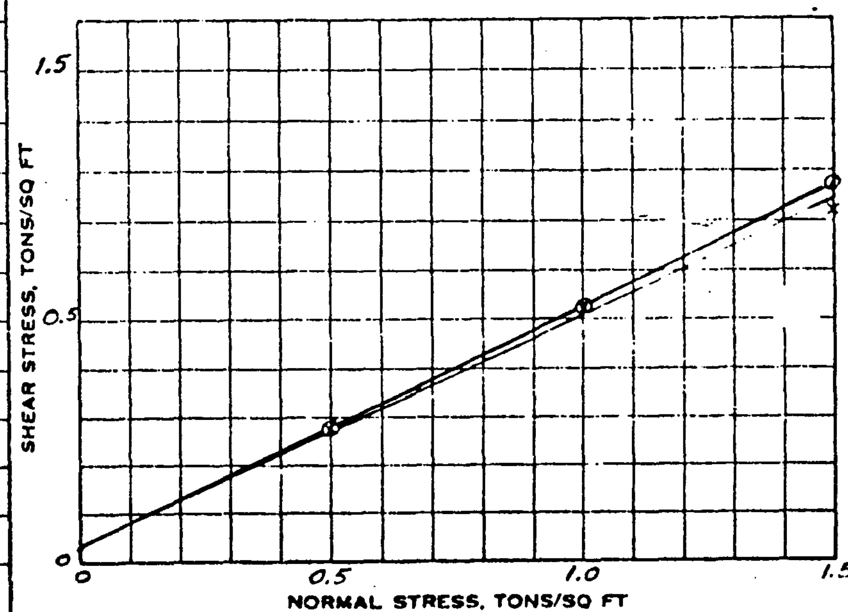
PLATE 7

ENG FORM 2081

ARMY - SFD - C. E., CALIF.







Classification	SANDY SILT (ML)			
LL	G 2.73			
PL N.P.	D_{10} 0.01 mm			
Test No.	A	B	C	
Initial Water Content, W_o %	12.7	14.4	15.5	
Dry Density Lbs/Cu Ft	84	84	87	
Void Ratio, e_o	1.033	1.026	0.966	
Saturation, S_o %	34	38	44	
Duration of Test Min.	3240	3210	3300	
WC after Test, W_f %	28.7	30.0	27.7	
Normal Stress, σ T/Sq Ft	0.50	1.00	1.50	
Max. Shear Stress, τ T/Sq Ft	0.28	0.52	0.77	
ULT " " " "	0.28	0.52	0.72	

Type of Test: Strain - Control; ✓ Consolidated; ✓ Drained	Type of Specimen: UNDISTURBED; 3.25 in. square; 1.0 in. thickness

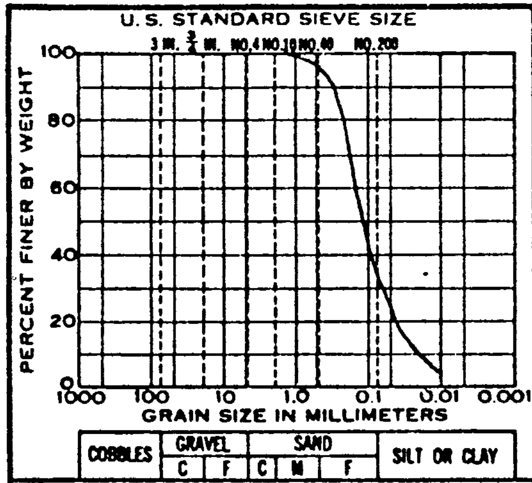
SHEAR VALUES	Ø	TAN Ø	C, T/SQ FT
Maximum	26°	0.488	0.03
Ultimate	25°	0.466	0.03

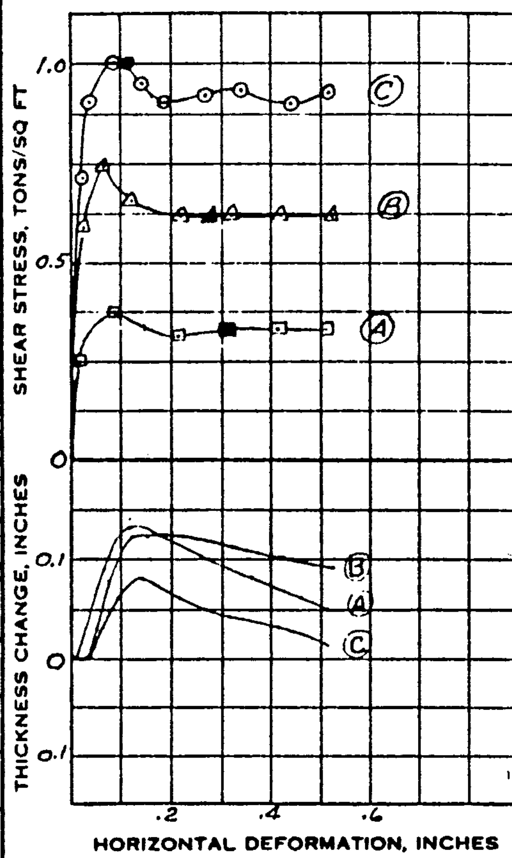
Remarks:

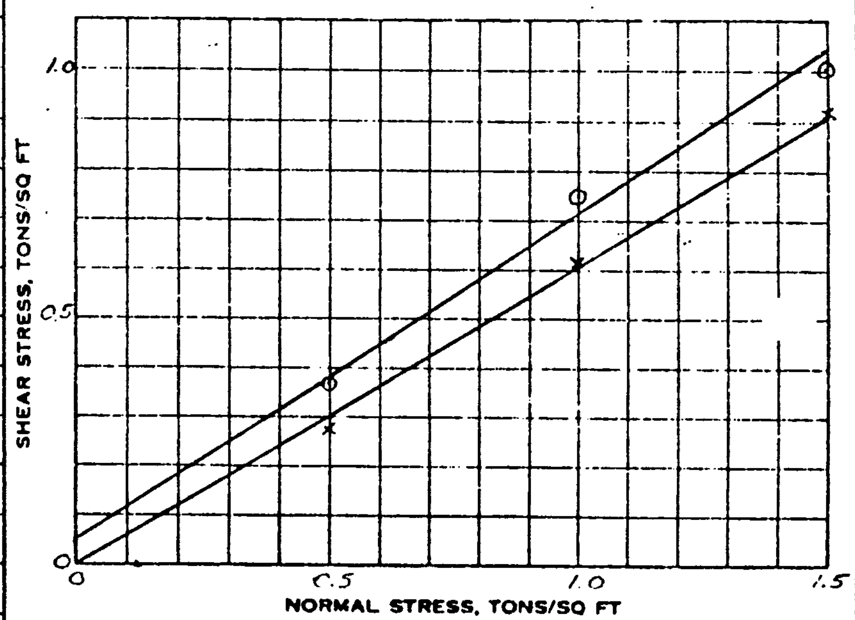
Project AMERICAN RIVER LEVEE	
Area	
Boring No. 2F-2	Sample No. 12438-SA
Elev or Depth 5.0'-6.0'	Date

DIRECT SHEAR TEST REPORT

PLATE 8







Classification: SILTY SAND (SM)

LL	G 2.70
PL N.P.	D_{10} 0.02 mm

Test No.	A	B	C	
Initial Water Content, W_0 %	14.9	14.9	14.9	
Dry Density Lbs/Cu Ft	104	105	105	
Void Ratio, e_0	.611	.608	.608	
Saturation, S_0 %	66	66	66	
Duration of Test MIN.	1630	1650	1650	
WC after Test, W_f %	21.2	19.8	20.9	
Normal Stress, σ T/Sq Ft	0.50	1.00	1.50	
Max. Shear Stress, τ T/Sq Ft	0.37	0.75	1.00	
ULT. " " "	0.28	0.62	0.92	

Type of Test	Type of Specimen
Strain – Control	REMOLDED
✓ Consolidated	3.25" in. square
✓ Drained	1.0 in. thickness

SHEAR VALUES	Ø	TAN Ø	C, T/SQ FT
Maximum	33°	0.650	0.04
Ultimate	31°	0.601	0

Remarks: REMOLDED TO 95% MAXIMUM DENSITY @ OPTIMUM MOISTURE.

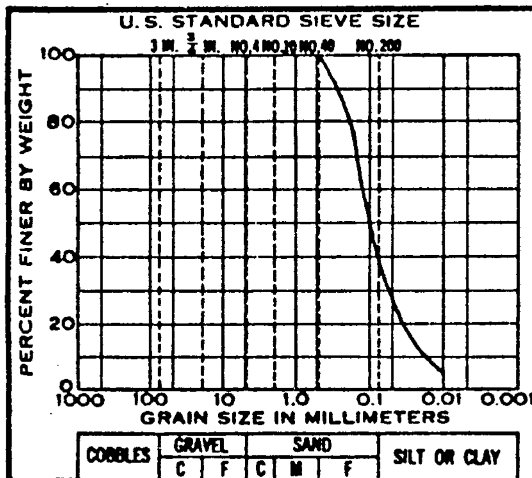
Project: AMERICAN RIVER LEVEE

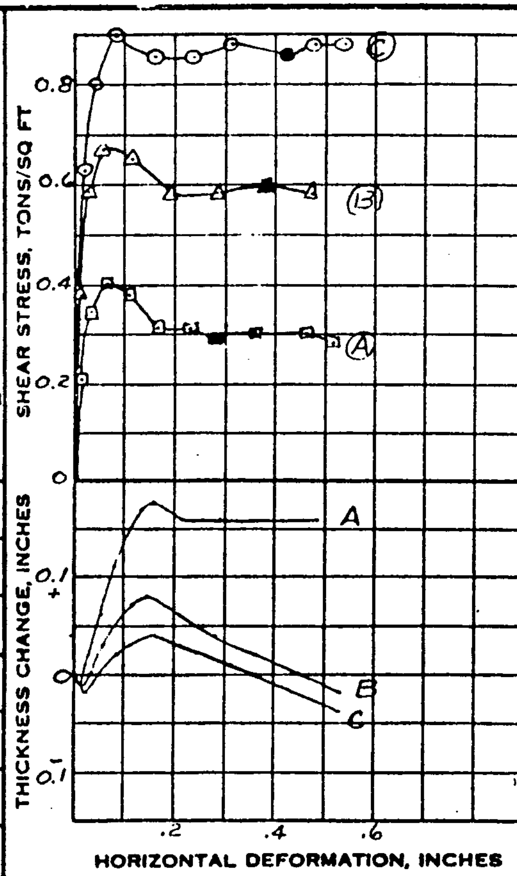
Area

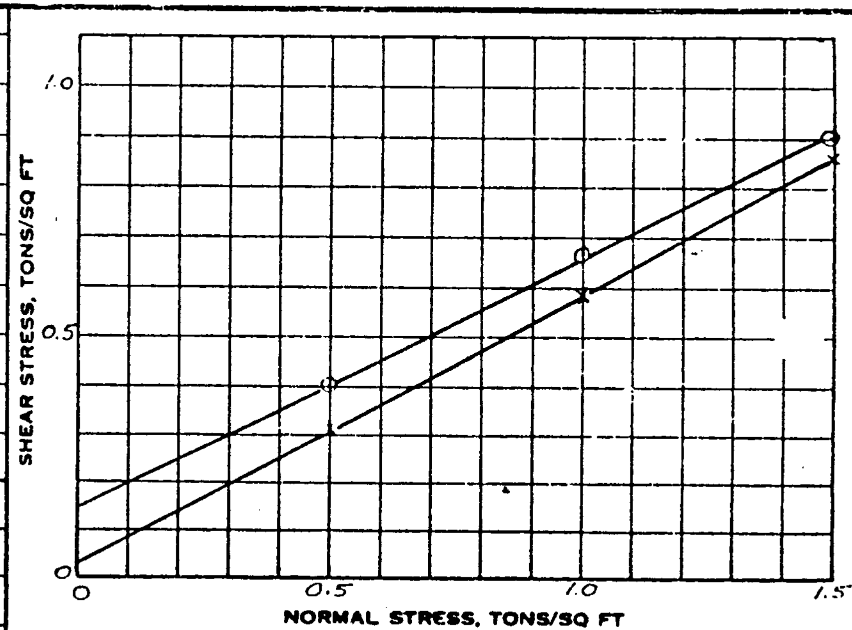
Boring No. 2B-2	Sample No. 12477-SA
Elev or Depth 0 - 6.0'	Date

DIRECT SHEAR TEST REPORT

2092







Classification	Silty Sand (SM)			
LL		G 2.73		
PL N.P		D_{10} 0.02 mm		
Test No.	A	B	C	
Initial Water Content, W_o %	20.6	20.9	21.1	
Dry Density Lbs/Cu Ft	92	92	92	
Void Ratio, e_o	.855	.859	.861	
Saturation, S_o %	66	66	67	
Duration of Test Min.	1590	1560	1650	
WC after Test, W_f %	27.1	27.7	26.9	
Normal Stress, σ T/Sq Ft	0.50	1.00	1.50	
Max. Shear Stress, τ T/Sq Ft	0.40	0.67	0.90	
Ult. " " "	0.31	0.58	0.84	

Type of Test Strain-Control	Type of Specimen Remolded
Consolidated	3.25 in. square
Drained	1.0 in. thickness

SHEAR VALUES	Ø	TAN Ø	C, T/SQ FT
Maximum	27°	0.510	0.15
Ultimate	29°	0.554	0.03

Remarks: Remolded to 95% Maximum Density @ Optimum Moisture

Project American River Levee	
Area	
Boring No. 2B-6	Sample No. 12482-SA
Elev or Depth 0.4' - 3.0'	Date

DIRECT SHEAR TEST REPORT

ENG FORM 1 DEC 51 2092

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PLATE 10

TEST DATA SUMMARY

PROJECT American River Levee FEATURE Pumping Station

HOLE NO.	DIV. SAMPLE NO.	DEPTH OR ELEV. OF SAMPLE	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} mm	ATTERBERG LIMITS LL	PL	SPECIFIC GRAVITY G	NATURAL WATER CONTENT %	NATURAL DRY DENS LBS/CU FT	COMPACTION DATA OPTIMUM WATER %	MAXIMUM DRY DENSITY LBS/CU. FT.	SHEAR DATA INITIAL e	DRY DENS LBS/CU FT	W_i %	W_f %	S_o %	TYPE TEST	SPECIMEN SIZE INCHES	TEST	σ_{iii} T/SQFT	σ_i T/SQ FT	C T/SQ FT	ϕ DEGREES	PERMEABILITY e	K FT/DAY	CONSOLIDATION DATA P_o T/SQ FT	P_c T/SQ FT	C_c	t	TYPE OF SAMPLE
5F-8	12407	0-1.0'	Silty Sand(SM)(V)								1.6	90-Vibrated																					6" [illegible]
"	"	"									1.6	74-Loose																					"
"	12408	1.5-2.5'	Silty Sand (SM)	0	88	12		—	NP		7.0	86-Top																					"
"	"	"									6.4	81-Top																					"
"	12409	3.0-4.0'	Silty Sand(SM)(V)								5.5	76-Bottom																					"
"	"	"									5.7	74-Bottom																					"
"	12410	4.5-5.5'	Silty Sand(SM)(V)								6.4	80-Top																					"
"	"	"									4.9	78-Middle																					"
"	12411	6.0'-7.0'	Silty Sand(SM)(V)								10.2	79-Top																					"
"	"	"									11.1	90-Middle																					"
"	12412	7.5'-8.5'	Silty Sand(SM)(V)								7.3	80-Top																					"
"	"	"									7.0	84-Middle																					"
"	12413	9.0'-10.0'	Silty Sand(SM)(V)								10.4	85-Top																					"
"	"	"	Sandy Silt (ML)	0	8	92	.006	37	28		37.2	81-Bottom																					"
"	12414	10.5'-11.5'	Sandy Silt(ML)(V)								37.6	77-Top																					"
"	"	"	Silty Sand(SM)(V)								7.1	89-Bottom																					"
"	12415	12.0'-13.0'	Sandy Silt (ML)	0	46	54	.008	23	20		20.0	92-Top																					"
"	"	"									25.2	90-Bottom																					"
"	12416	13.5'-14.5'	Silty Sand(SM)(V)								13.6	96-Top																					"
"	"	"									19.7	83-Bottom																					"
"	12417	15.0'-16.0'	Silty Sand (SM)	0	82	18		-	NP		16.3	78-Top																					"
"	"	"	Sandy Silt(ML)(V)								14.7	71-Bottom																					"

(V) Visual Classification

SPECIFIC GRAVITY
(a) = Minus No. 4
(b) = Plus No. 4

COMPACTION DATA
(1) = Standard AASHO
(2) = Modification (See Text)
(3) = ~~Maximum-Minimum~~ (Vibrated Maximum)

TC - Triaxial Compression
UC - Unconfined Compression

DS - Direct Shear
UU - Unconsolidated Undrained

CD - Consolidated Drained
CU - Consolidated Undrained

TEST DATA SUMMARY

PROJECT American River Levee FEATURE Pumping Station

HOLE NO.	DIV. SAMPLE NO.	DEPTH OR ELEV. OF SAMPLE	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} mm	ATTERBERG LIMITS LL	PL	SPECIFIC GRAVITY G	NATURAL WATER CONTENT %	NATURAL DRY DENS LBS/CU FT	COMPACTION DATA OPTIMUM WATER %	MAXIMUM DRY DENSITY LBS/CU. FT.	INITIAL e	SHEAR DATA DRY DENS LBS/CU FT	W_1 %	W_P %	S_0 %	TYPE TEST	SPECIMEN SIZE INCHES	TEST	σ_{III} T/SQ FT	σ_I T/SQ FT	C T/SQ FT	ø DEGREES	PERMEABILITY e	K FT/DAY	CONSOLIDATION DATA P_0 T/SQ FT	P_C T/SQ FT	C_c	t	TYPE OF SAMPLE
5F-8	12407	0-1.0'	Silty Sand(SM)(V)								1.6	90-Vibrated																					6" Cored
"	"	"									1.6	74-Loose																					"
"	12408	1.5-2.5'	Silty Sand (SM)	0	88	12		—	NP		7.0	86-Top																					"
"	"	"									6.4	81-Top																					"
"	12409	3.0-4.0'	Silty Sand(SM)(V)								5.5	76-Bottom																					"
"	"	"									5.7	74-Bottom																					"
"	12410	4.5-5.5'	Silty Sand(SM)(V)								6.4	80-Top																					"
"	"	"									4.9	78-Middle																					"
"	12411	6.0-7.0'	Silty Sand(SM)(V)								10.2	79-Top																					"
"	"	"									11.1	90-Middle																					"
"	12412	7.5-8.5'	Silty Sand(SM)(V)								7.3	80-Top																					"
"	"	"									7.0	84-Middle																					"
"	12413	9.0'-10.0'	Silty Sand(SM)(V)								10.4	85-Top																					"
"	"	"	Sandy Silt (ML)	0	8	92	.006	37	28		37.2	81-Bottom																					"
"	12414	10.5'-11.5'	Sandy Silt(ML)(V)								37.6	77-Top																					"
"	"	"	Silty Sand(SM)(V)								7.1	89-Bottom																					"
"	12415	12.0-13.0'	Sandy Silt (ML)	0	46	54	.008	23	20		20.0	92-Top																					"
"	"	"									25.2	90-Bottom																					"
"	12416	13.5-14.5'	Silty Sand(SM)(V)								13.6	96-Top																					"
"	"	"									19.7	83-Bottom																					"
"	12417	15.0-16.0'	Silty Sand (SM)	0	82	18		-	NP		16.3	78-Top																					"
"	"	"	Sandy Silt(ML)(V)								14.7	71-Bottom																					"

(V) Visual Classification

SPECIFIC GRAVITY
(a) = Minus No. 4
(b) = Plus No. 4

COMPACTION DATA
(1) = Standard AASHO
(2) = Modification (See Text)
(3) = Maximum—Minimum (Vibrated Maximum)

TC – Triaxial Compression
UC – Unconfined Compression
DS – Direct Shear
UU – Unconsolidated Undrained
CD – Consolidated Drained
CU – Consolidated Undrained

TEST DATA SUMMARY

PROJECT **American River** FEATURE **Levee**

HOLE NO.	DIV. SAMPLE NO.	DEPTH OR ELEV. OF SAMPLE	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} mm	ATTERBERG LIMITS LL	PL	SPECIFIC GRAVITY G	NATURAL WATER CONTENT %	NATURAL DRY DENS LBS/CU FT	COMPACTION DATA OPTIMUM WATER %	MAXIMUM DRY DENSITY LBS/CU. FT.	SHEAR DATA INITIAL e	DRY DENS LBS/CU FT	W_i %	W_f %	S_o %	TYPE TEST	SPECIMEN SIZE INCHES	TEST	σ_{III} T/SQ FT	σ_I T/SQ FT	C T/SQ FT	ϕ DEGREES	PERMEABILITY e	K FT/DAY	CONSOLIDATION DATA P_o T/SQ FT	P_c T/SQ FT	C_c	t	Type of Sample
2F-2	12438	5.0-6.0'	Sandy Silt (ML)	0	38	62	.01	-	NP	2.73					1.083	84	12.7	28.7	34	D.S.	3½X3½X1	CD		0.5	.03	26(Max)							6" Cored
															1.026	84	14.4	30.0	38	"	"	"		1.0	.03	25(Ult)							
															.966	87	15.5	27.7	44	"	"	"		1.5									
2B-2	12477	0-6.0'	Silty Sand (SM)	0	66	34	.02	-	NP	2.70			15.3	110.0(1)	.611	104	14.9	21.2	66	DS	3½X3½X1	CD		0.5	.04	33(Max)							Sack
															.608	105	14.9	19.8	66	"	"	"		1.0	0	31 (Ult)							
															.608	105	14.8	20.9	66	"	"	"		1.5									
3-6	12482	0.4-3.0	Silty Sand (SM)	0	58	42	.02	-	NP	2.73			21.0	96.2(1)	.855	92	20.6	27.1	66	DS	9½X3½X1	CD		0.5	.15	27 (Max)							Sack
															.859	92	20.9	27.7	66	"	"	"		1.0	.05	29 (Ult)							
															.861	92	21.1	26.9	67	"	"	"		1.5									

SPECIFIC GRAVITY
(a) = Minus No. 4
(b) = Plus No. 4

COMPACTION DATA
(1) = Standard AASHO
(2) = Modification (See Text)
(3) = Maximum-Minimum (Vibrated Maximum)

TC - Triaxial Compression
UC - Unconfined Compression

DS - Direct Shear
UU - Unconsolidated Undrained

CD - Consolidated Drained
CU - Consolidated Undrained

SAMPLE LOG

DISTRICT: Sacramento | PROJECT: American River Levee | HOLE NO.: 5F-8

REMARKS: 6" Cored Samples | SHEET 1 OF 2

DIV. NO.	F.S. NO.	DEPTH (FT.)	TYPE & CONDITION OF SAMPLE, REMARKS	SYMBOL	CLASSIFICATION OF SOIL	FIELD MOISTURE
		0				
12407-SA C-1506	1		Tan, dry, loose fine sand. Hair roots. Non-plastic	SM	Silty Sand	
13408-SA C-1507	2	2	Tan, damp semi-compact sand. Non-plastic	SM	Silty Sand	
		3				
13409-SA C-1508	3		Same as above.	SM	Silty Sand	
		4				
13410-SA C-1509	4	5	Top 3": Very light tan, loose, Non-plastic. Bottom: Mixture of tan and light tan loose sand.	SM	Silty Sand	
		6				
13411-SA C-1510	5		Light brown, damp, semi-compact sand. Non-plastic.	SM	Silty Sand	
		7				
13412-SA C-1511	6	8	Similar to above. Thin layer of white sand.	SM	Silty Sand	

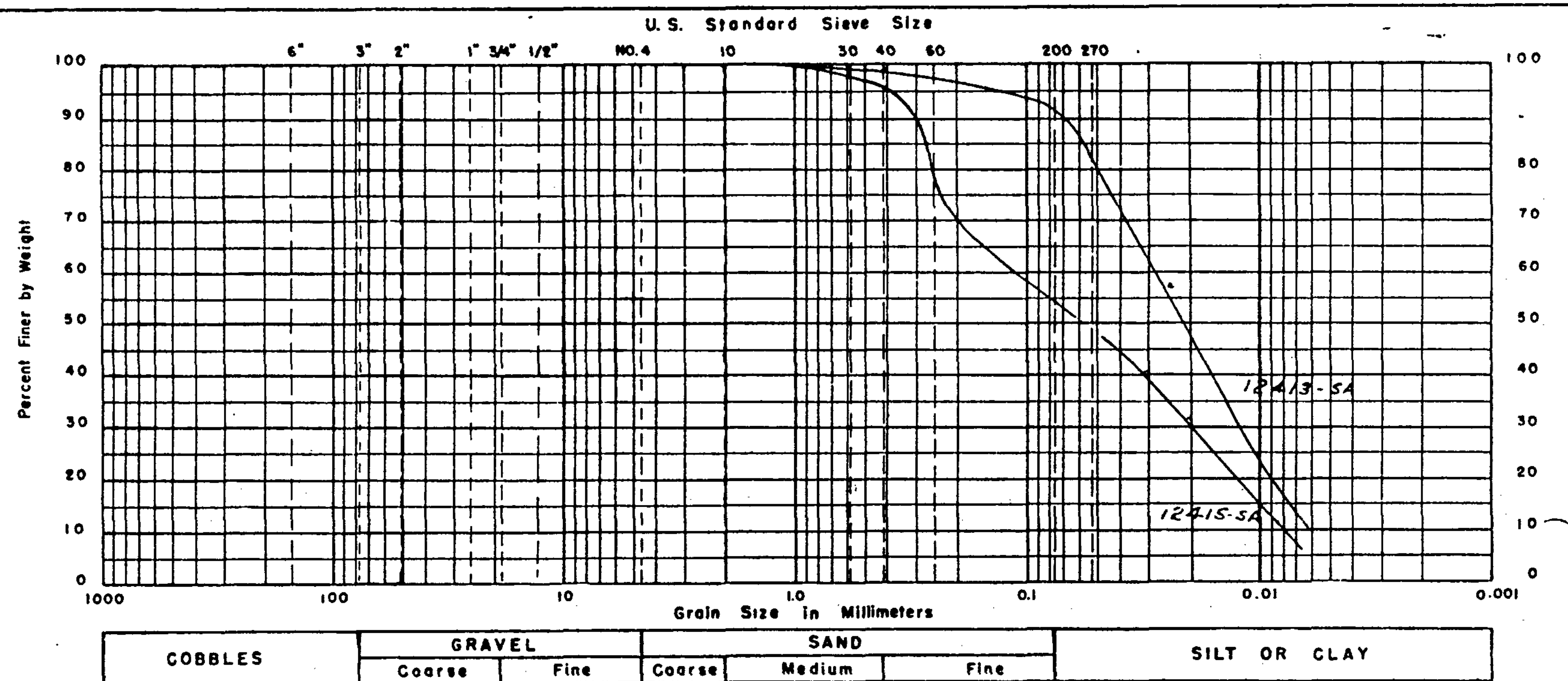
SAMPLE LOG

DISTRICT: Sacramento	PROJECT: American River Levee	HOLE NO.: 5F-8
REMARKS:	6" Cored Samples	SHEET 2 OF 2

[illegible] NO.	F.S. NO.	DEPTH (FT)	TYPE & CONDITION OF SAMPLE, REMARKS	SYMBOL	CLASSIFICATION OF SOIL	FIELD MOISTURE
		9				
12413-SA C-1512	7		Top 85% light brown, damp, semi-compact sand Bottom: Moist brown silt with sand pockets	SM ML	Silty Sand Sandy Silt	
		10				
12414-SA C-1513	8	11	Top 50% stratified sand and silt Bottom 50%: Light brown semi-compact, damp sand	ML SM	Sandy Silt Silty Sand	
		12				
12415-SA C-1514	9		Top: Moist, soft stratified sand and silt. Bottom: brown, moist with layers of gray silt and brown silt	ML	Sandy Silt	
		13				
12416-SA C-1515	10	14	Silty sand with layers of white sand, gray-brown silt and dark brown silt.	SM	Silty Sand	
		15				
12417-SA C-1516	11		Top: Brown, moist, loose sand. Appeared disturbed. Bottom: Dark brown moist silt.	SM ML	Silty Sand Sandy Silt	
		16				

CORPS OF ENGINEERS, U.S. ARMY
SOUTH PACIFIC DIVISION LABORATORY
SAUSALITO, CALIFORNIA

MECHANICAL ANALYSIS PLOT



DIV. NO.	DIST. NO.	F.S. NO.	LL	PI	SOIL NAME
12413-SA	C-1512-56	7	37	9	SANDY SILT (ML)
12415-SA	C-1514-56	9	23	3	" " "

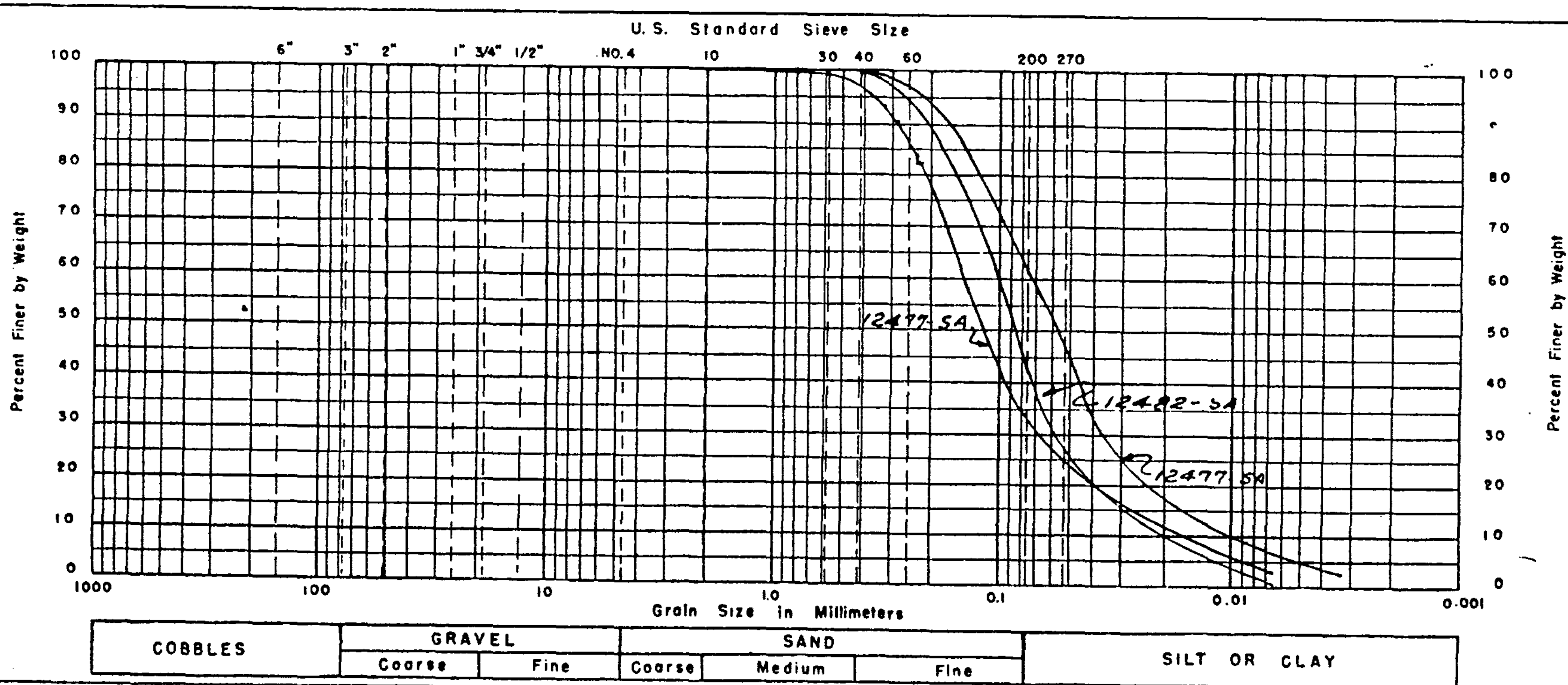
DISTRICT: SACRAMENTO
PROJECT: AMERICAN RIVER LEVEE
HOLE NO: 5F-8
STATION:
REMARKS: PUMPING STATION

TESTED	COMPUTED	DRAWN	CHECKED
HD	H.D.	EBH.	D.O.P.

SPD Form 175 CIVIL
24 JAN 57

CORPS OF ENGINEERS, U.S. ARMY
SOUTH PACIFIC DIVISION LABORATORY
SAUSALITO, CALIFORNIA

MECHANICAL ANALYSIS PLOT



DIV. NO.	DIST. NO.	HOLE NO.	LL	PI	SOIL NAME
12438-SA	C-1418-56	2F-2		NP	SANDY SILT (ML)
12477-SA	C-1433-56	2B-2		N.P	SILTY SAND (SM)
12482-SA	C-1473-56	2B-6		NP	" " "

DISTRICT: SACRAMENTO
PROJECT: AMERICAN RIVER LEVEE
HOLE NO:
STATION:
REMARKS:

TESTED	COMPUTED	DRAWN	CHECKED
H.D.	H.D.	E.B.H.	D.O.P.

SPD Form 175 Civil
24 JAN 52

APPENDIX M-2-C

REPORT OF SOIL TESTS - PL84-99 POST FLOOD REHABILITATION - JUNE 1986 (EXCERPTS)

REPORT
OF
SOIL TESTS

PL 84-99 POST FLOOD REHABILITATION

July 1986

AUTHORIZATION

1. Results of tests reported herein were requested by the Sacramento District in laboratory request No. SPKED-D-86-42 dated 14 May 1986.

SAMPLES

2. Disturbed samples in plastic bags were received during the period 5 June and 24 June 1986. Identification of samples is on the Soil Test Result Summary, plates 1 to 16.

TESTING PROGRAM

3. The program was in accordance with the test request. Tests included Sieve Analysis and Atterberg Limits.

TEST METHODS

4. a. Grain-Size Analysis, Atterberg Limits. Testing methods conformed to the procedures described in Engineer Manual, EM 1110-2-1906, "Laboratory Soil Testing", 30 November 1970.

b. Classification. The soils were classified in accordance with the "Unified Soil Classification System", TM 3-337, Appendix A, April 1960, reprinted May 1967.

RESULTS

5. Results of tests are shown on the following plates:

Subject	Plate No.
Soil Test Result Summary:	
RD 1000 Garden Highway Levee	1-4 *
Yuba River, RD 784	5-8 *
DWR Maintenance Area 9	9-11 *
American River, Left Bank Levee	12-13
Yolo Bypass, RD 785, RD 827, & RD 1600	14-16 *

* Not relevant to this study and therefore not included in Appendix C

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT PL 84-99 Post Flood Rehabilitation - American River, ~~Left~~ Right Bank Levee DATE 24 June 1986

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation		Laboratory Descriptive Classification	Mechanical Analysis-% Finer											Liquid Limit	Plasticity Index	Field Moist. %
						Gravel					Sand					Fines			
			From	To							#4	#10	#40	#60	#100	#200			
96848	F-1-A	2	3	4	Sandy Silty Clay (CL-ML)							100	98	97	93	79	26	6	
96849		8	12	13.5								100	99	98	95	78			
96853		16	24	25.5	Clayey Silt (ML							100	95	95	94	86	31	6	
96855		21	31.5	33	Sandy Silt (ML)							100	99	98	93	64		NP	
96850	F-2-A	3	6	7.5								100	99	99	96	76			
96851		9	13.5	15	Sandy Silt (ML)							100	99	99	96	81	25	1	
96852		15	22.5	24								100	99	99	97	83			
96854		19	28.5	30	Silt (ML)							100	99	98	97	86		NP	
96856	F-3-A	4	2.5	6	Sandy Silt (ML)							100	99	98	95	81	27	2	
96857		10	13.5	15	Sandy Silt (ML)							100	99	99	97	84	27	2	
96858		17	24	25.5	Silt (ML)							100	98	98	96	86	28	2	
96859		20	25.5	30								100	98	98	95	73			

SPD Form 66A
1 May 83

plate 12

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT PL 84-99 Post Flood Rehabilitation - American River, Left Bank Levee DATE 24 June 1986

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel					Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
96860	F-4-A	5	6	7								100	99	94	83	64	–		
96861		13	18	19.5	Sandy Silt (ML)							100	99	99	98	85	32	5	
96862		18	22.5	27								100	98	97	96	86			
96863		26	37.5	39	Silt (ML)							100	99	99	98	86	28	2	

SPD Form 66A
1 May 83

plate 13

APPENDIX M-2-D

REPORT OF SOIL TESTS - SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION - JUNE 1987 (EXCERPTS)

REPORT
OF
SOIL TESTS

SACRAMENTO RIVER FLOOD
CONTROL SYSTEM EVALUATION

JULY 1987

AUTHORIZATION

Testing services are authorized by the Sacramento District per DA Form 2544 Nos SPKED-F-87-60 dated 15 Apil 1987, and change orders 1, 2, and 3 dated 29 April, 4 May, and 14 May 1987.

SAMPLES

On 17 and 29 April 1987,and 13 May 1987, five hundred thirty-six samples in plastic bags and eighty-one samples in 3-inch tubes were received at the Laboratory. Identification of the samples which were tested are shown on the Soil Test Result Summary plates

TESTING PROGRAM

The program was in accordance with the test request as per SPD Form 29. dated 1, 5 and 13 May 1987.

TESTS AND TEST METHODS

Grain-size Analysis, Atterberg Limits, Field Unit Weight, Specific Gravity, Organic Content, Triaxial Compression, and Consolidation. Testing conformed to the procedure described in Engineer Manual, "EM 1110-2-1906, Laboratory Soil Testing," 30 November 1970.

CLASSIFICATION

RESULTS

Results of tests are shown on the following plates:

Subject	Plate No
Soil Test Result Summary	1 - 20
Field Unit Weight Summary	21 - 22
Triaxial Compression Test Report	
"Q" Tests	23 - 43
Consolidated Test Report	44 - 45

NOTE: Only the plates relevant to the American River study are included in Appendix D

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

Contract #2 ARFCD &MA 10 & 11

SOIL TEST RESULT SUMMARY

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION

DATE June 1987

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation		Laboratory Descriptive Classification	Mechanical Analysis-% Finer											Liquid Limit	Plasticity Index	Field Moist. %
						Gravel					Sand					Fines			
			From	To				3/4	1/2	3/8	#4	#10	#40	#60	#100	#200			
101517	DH-1	B-1	0.5	3.5	** Sandy Silt(ML)							100	98	94	79	53	-	NP	
101518	"	B-2	6.5	9.5	Sandy Silt(ML)				100	98	98	98	94	89	78	53		NP	7.0
101519	DH-2	B-1	0.5	3.5	Silty Sand(SM)					100	99	99	98	97	85	50			15.4
101521	"	S-1	9.5	11.7	** Silty Sand(SM)							100	99	94	77	47		NP	
101522	DH-3	B-1	1.5	3.0	Silty Sand (SM)							100	98	78	49	28			
101523	"	B-2	7.5	10.5	Silty Sand(SM)							100	98	88	66	38			
101524	DH-4	B-1	0.5	3.5	Silty Sand(SM)			100	99	98	97	96	92	78	58	35			10.3
101526	"	B-3	18.5	21.5	Silty Sand (SM)							100	99	81	42	17			
101528	DH-4A	S-1	6	8.2	Silty Sand (SP-SM)							100	95	55	21	8		NP	

SPD Form 66A
1 May 83

PLATE 7, CONT. H2

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

Contract #2 ARFCD & MA 10 & 11

SOIL TEST RESULT SUMMARY

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION

DATE June 1987

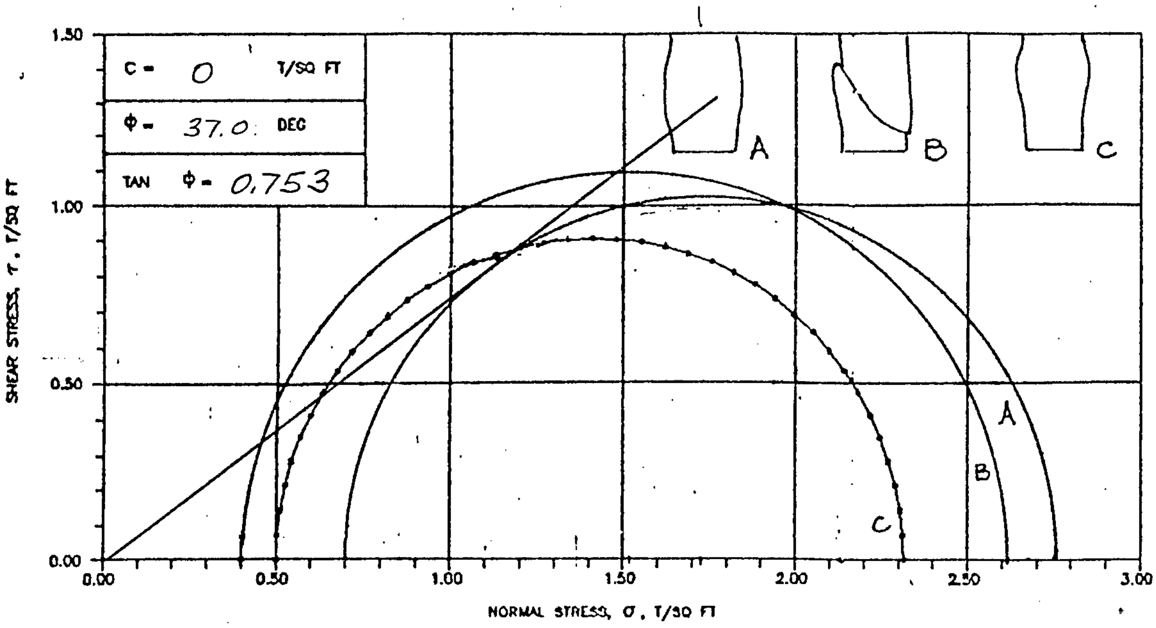
Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer	Gravel 1	Gravel 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
101675	DH-28A	B-2	6	9	Sandy Silt (ML)								100	96	79	54-	--		
101677	"	S-1	16.5	17.7	*Silt(ML)							100	99	99	97	94		NP	
101678	DH-29	B-1	6.5	9.5	Silty Sand (SP-SM)		100	99	97	97	96	94	68	29	17	11			3.5
101679	"	B-2	14	17	Sandy Silt(ML)							100	99	97	88	66		NP	12.0
101680	DH-30	B-1	3.5	6.5	Sandy Silt(ML)							100	98	93	80	56		NP	
101681	"	B-2	12.5	15.5	Sandy Silt (ML)							100	98	94	79	53			
101682	DH-31	B-1	2	5	Sandy Silt(ML)							100	99	97	90	69	23	2	
101684	"	S-1	12.5	14.7	**Sandy Silt(ML)								100	98	91	69		NP	

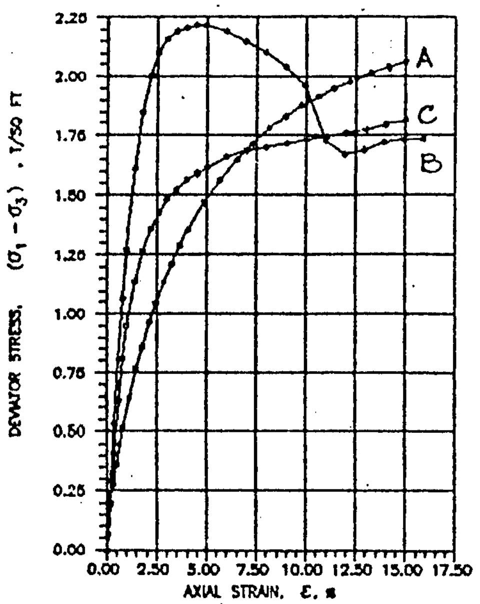
COMMENTS: * Very low plasticity makes the accuracy of the plastic limit rollset doubtful e.g. 101654 showed a LL of 27 and a P.T. of -0.5.

** Called "NP" due to very marginal plasticity which would have yielded highly dubious results in plastic limits rollout.

SPD Form 66A
1 May 83

PLATE 15, CONT.#2





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	W_o	10.4	9.4	28.3
	DRY DENSITY LB/CU FT	γ_{d_o}	88.4	90.1	88.1
	SATURATION, %	S_o	31	30	84
	VOID RATIO	e_o	0.906	0.836	0.918
BEFORE SHEAR	WATER CONTENT, %	W_c	10.4	9.4	28.3
	DRY DENSITY LB/CU FT	γ_{d_c}	90.4	90.9	89.5
	SATURATION, %	S_c	33	30	87
	VOID RATIO	e_c	0.863	0.820	0.888
	FINAL BACK PRESSURE, T/SQ FT	u_b	–	–	–
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.70	0.40	0.50
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.05	2.22	1.82
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	15	6	16
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	1.67	–
INITIAL DIAMETER, IN.		D_o	2.87	2.89	2.84
INITIAL HEIGHT, IN.		H_o	6.26	6.26	6.26

CONTROLLED STRAIN TEST Yes

DESCRIPTION OF SPECIMEN See Test Summary (Silty Sand)

LL	PL	PI	G_S 2.70	TYPE OF SPECIMEN Undisturbed	TYPE OF TEST Q

REMARKS

	Hole No./F.S. No.	Depth	Div. No.
A	DH2/S-1	11.1'-11.7'	101521
B	DH4A/S-1	7.8'-8.2'	101528
C	DH13/S-1	8.9'-9.5'	101570

PROJECT Sacramento River Flood Control Study Evaluation

BORING NO. SAMPLE NO.

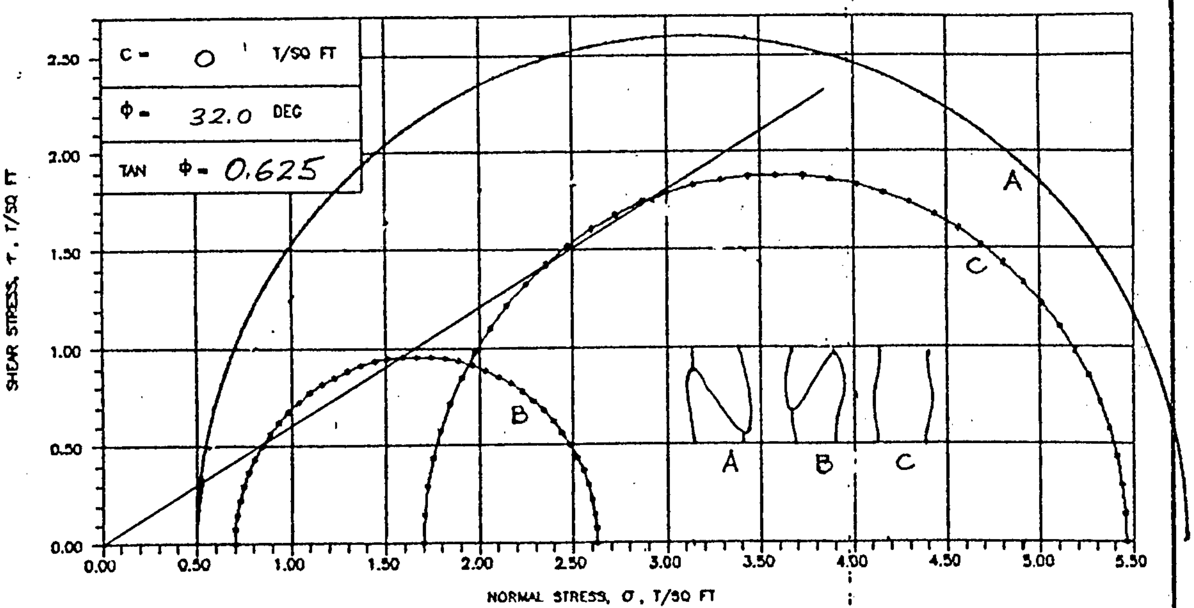
Contract #2 ARFCD & MA 10 & 11

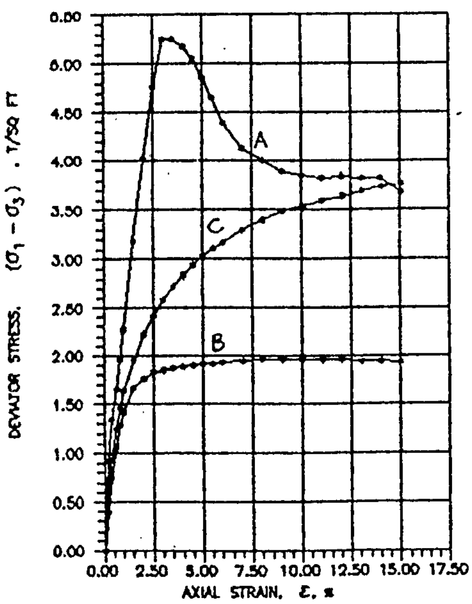
LABORATORY SPDED-GL DATE June 1987

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV. JUNE 1970

PLATE 31, CONT. #2





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	W_o	17.5	19.9	15.4
	DRY DENSITY LB/CU FT	γ_{d_o}	98.3	104.9	81.9
	SATURATION, %	S_o	66	89	39
	VOID RATIO	e_o	0.714	0.606	1.056
BEFORE SHEAR	WATER CONTENT, %	W_c	17.5	19.9	15.4
	DRY DENSITY LB/CU FT	γ_{d_c}	99.3	105.5	84.7
	SATURATION, %	S_c	68	90	42
	VOID RATIO	e_c	0.696	0.597	0.988
	FINAL BACK PRESSURE, T/SQ FT	U_o	–	–	–
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.50	0.70	1.70
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	5.26	1.96	3.76
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	5	9	16
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–
INITIAL DIAMETER, IN.		D_o	2.88	2.89	2.88
INITIAL HEIGHT, IN.		H_o	6.26	6.26	6.26

CONTROLLED STRAIN TEST Yes

DESCRIPTION OF SPECIMEN *Sandy Silt - DH-25, Clay (CL) - DH-20, Sandy Clayey Silt (ML) DH-19A*

LL	PL	PI	G_s 2.70	TYPE OF SPECIMEN Undisturbed	TYPE OF TEST Q

REMARKS

	HoleNo.	F.S.No.	Depth	Div. No
A	DH-19A	S-1	7.0'-7.5'	101628
B	DH-20	S-1	12.0'-12.5'	101634
C	SH 25	S-1	30.8'-31.3'	101660

PROJECT Sac. River Flood Control Study Evaluation

BORING NO. SAMPLE NO.

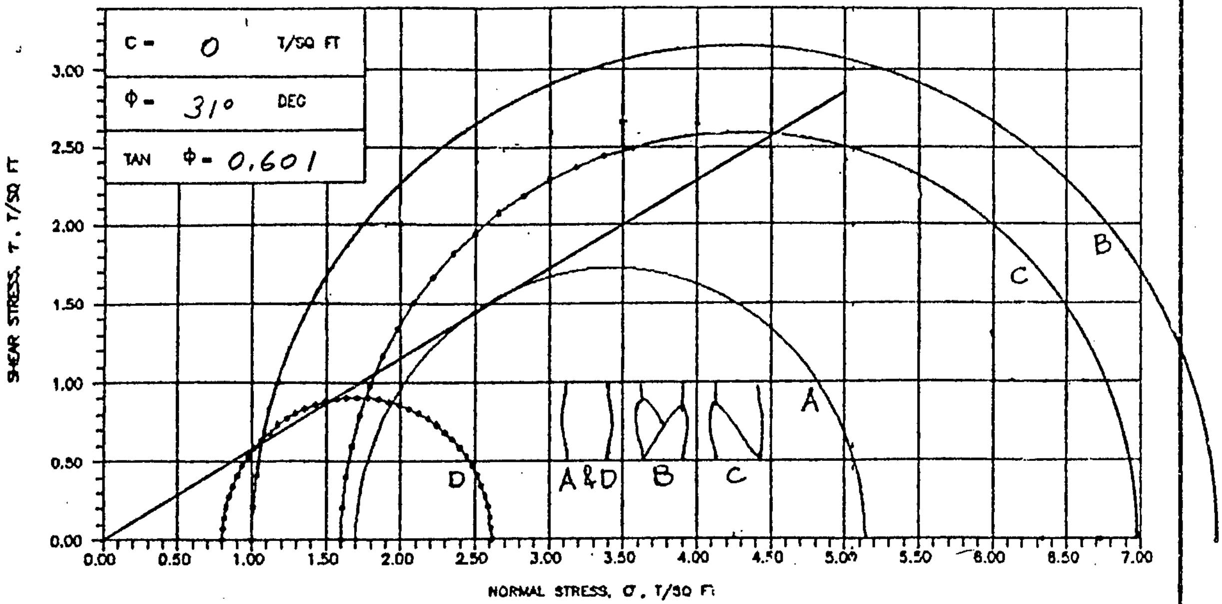
Contract #2 ARFCD & MA 10 & 11

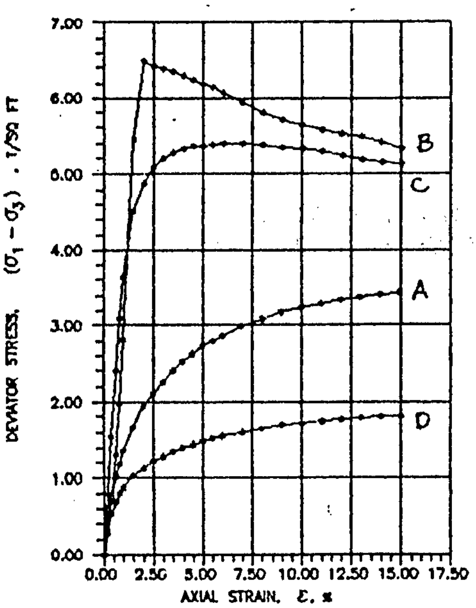
LABORATORY SPD LAB DATE June 1987

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV. JUNE 1970

PLATE 34, CONT. #2





			A	B	C	D
SPECIMEN NO.			A	B	C	D
INITIAL	WATER CONTENT, %	w_o	19.3	34.7	19.6	11.5
INITIAL	DRY DENSITY LB/CU FT	γ_{d_o}	87.2	77.7	96.9	88.3
INITIAL	SATURATION, %	S_o	56	80	72	34
INITIAL	VOID RATIO	e_o	0.932	1.169	0.739	0.908
BEFORE SHEAR	WATER CONTENT, %	w_o	19.3	34.7	19.6	11.5
BEFORE SHEAR	DRY DENSITY LB/CU FT	γ_{d_c}	89.6	78.9	99.0	90.0
BEFORE SHEAR	SATURATION, %	S_c	59	83	75	36
BEFORE SHEAR	VOID RATIO	e_c	0.880	1.136	0.702	0.871
BEFORE SHEAR	FINAL BACK PRESSURE, T/SQ FT	U_b	–	–	–	–
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.70	1.00	1.60	0.80
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	3.44	6.49	5.39	1.81
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	16	4	7	16
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–	–
INITIAL DIAMETER, IN.		D_o	2.87	2.87	2.86	2.84
INITIAL HEIGHT, IN.		H_o	6.28	6.26	6.26	6.26

CONTROLLED STRAIN TEST Yes

DESCRIPTION OF SPECIMEN See Test Summary

LL | PL | PI | G_S 2.70 | TYPE OF SPECIMEN Undisturbed | TYPE OF TEST Q

REMARKS

	Hole No.	FSNo.	Depth	Div. No.
A	DH-27	S-1	30.8'-31.3'	101667
B	DH-28A	S-1	17.1'-17.7'	101677
C	DH-28	S-1	29.1'-29.7'	101673
D	DH-31	S-1	14.2'-14.7'	101684

PROJECT Sac. River Flood Control Study Evaluation

BORING NO. | SAMPLE NO.

Contract #2 ARFCD & MA 10 & 11

LABORATORY SPD Lab | DATE June 1987

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV. JUNE 1970

APPENDIX M-2-E

REPORT OF SOIL TESTS - SACRAMENTO RIVER FLOOD CONTROL SUSTEM EVALUATION (AMERICAN RIVER) - APRIL 1988

REPORT
OF
SOIL TESTS

SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION
(AMERICAN RIVER)

April 1988

AUTHORIZATION

1. Testing services were authorized by the Sacramento District per DA Form 2544 No. CESPK-ED-G-87-93 dated 31 Aug 1987.

SAMPLES

2. On 31 August 1987, fifty pounds of samples in plastic bags were received at the Laboratory. Identification of the samples is shown on the Soil Test Result Summary plates.

TESTING PROGRAM

3. The program was in accordance with the test request as per Forms 29.

TESTS AND TESTS METHODS

4. Soil tests performed were Grain-size Analysis, Atterberg Limits, Specific Gravity, Triaxial Compression, Compaction, Diret Shear and Permeability. Testing conformed to the procedures described in Engineer Manual (EM) 1110-2-1906, "Laboratory Soil Testing," 30 November 1970.

CLASSIFICATION

5. Laboratory soil classifications conform to the "Unified Soil Classification System," Technical Manual (TM) 3-357, Appendix A, April 1960.

RESULTS

6. Results of tests are shown on the following plates:

SUBJECT	PLATE NO.
Soil Test Result Summary	1-2
Permeability	3&3A
Compaction	4-5
Triaxial Compression Test Reports "R"	6-17
Direct Shear Test Report	18-23

COMMENTS

1. Only those samples tested are identified in the referenced plates.

2. Samples will be disposed of six months following the date of this report.

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Sacramento River Flood Control System (American River) DATE: JANUARY 1988

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer Gravel 11/2	1	3/4	1/2	3/8	Sand #4	#10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
103964	4F-87-1A	1A	0.0	2.0	Silty Sand (SM)					100	99	99	98	84	58	31			1.5
103965	4F-87-1B	1B	0.0	3.0	Silty Sand (SM)					100	99	99	96	81	52	29			1.7
103966	4F-87-2A	2A	0.0	3.0	Silty Sand (SM)							100	97	84	64	43			2.0
103967	4F-87-2B	2B	0.0	3.0	Sandy Silt (ML)							100	99	95	81	56			2.0
103968	4F-87-3A	3A	0.0	3.0	Sandy Silt (ML)							100	99	97	84	59			2.3
103969	4F-87-3B	3B	0.0	4.0	Silty Sand (SM)							100	99	91	73	50			2.4
103970	4F-87-4A	4A	0.0	3.0	Silty Sand (SM)								100	99	97	87			2.9
103971	4F-87-4B	4B	0.0	4.0	Silt (ML)										100	96			3.7
103972	4F-87-5B	5B	0.0	3.0	Silty Sand (SM)							100	99	98	96	78			4.0
103973	4F-87-6A	6A	0.0	4.0	Silty Sand (SM-SP)					100	99	98	58	16	9	6			0.8
103974	4F-87-6B	6B	0.0	4.0	Silty Sand (SP)	100	99	99	99	99	99	99	93	87	73	41			1.7

SPD FORM 66A

PLATE I

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Sacramento River Flood Control System (American River) DATE: JANUARY 1988

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer Gravel 11/2	1	3/4	1/2	3/8	Sand #4	#10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
" COMPOSITE A"																			
103964	4F-87-1A	1A																	
103965	4F-87-1B	1B																	
103966	4F-87-2A	2A																	
103967	4F-87-2B	2B			Silty Sand (SM)	Sp.Gr. = 2.76						100	91	76	55	31		NP	
103968	4F-87-3A	3A																	
103969	4F-87-3B	3B																	
103973	4F-87-6A	6A																	
103974	4F-87-6B	6B																	
" COMPOSITE B "																			
103970	4F-87-4A	4A																	
103971	4F-87-4B	4B			Sandy Silt (ML)	Sp.Gr. = 2.76							100	98	96	77	31	4	
103972	4F-87-5B	5B																	

SPD-FORM 66A

PLATE2

Field Moisture Content

American River Geotechnical Investigations
Sacramento

November 1987

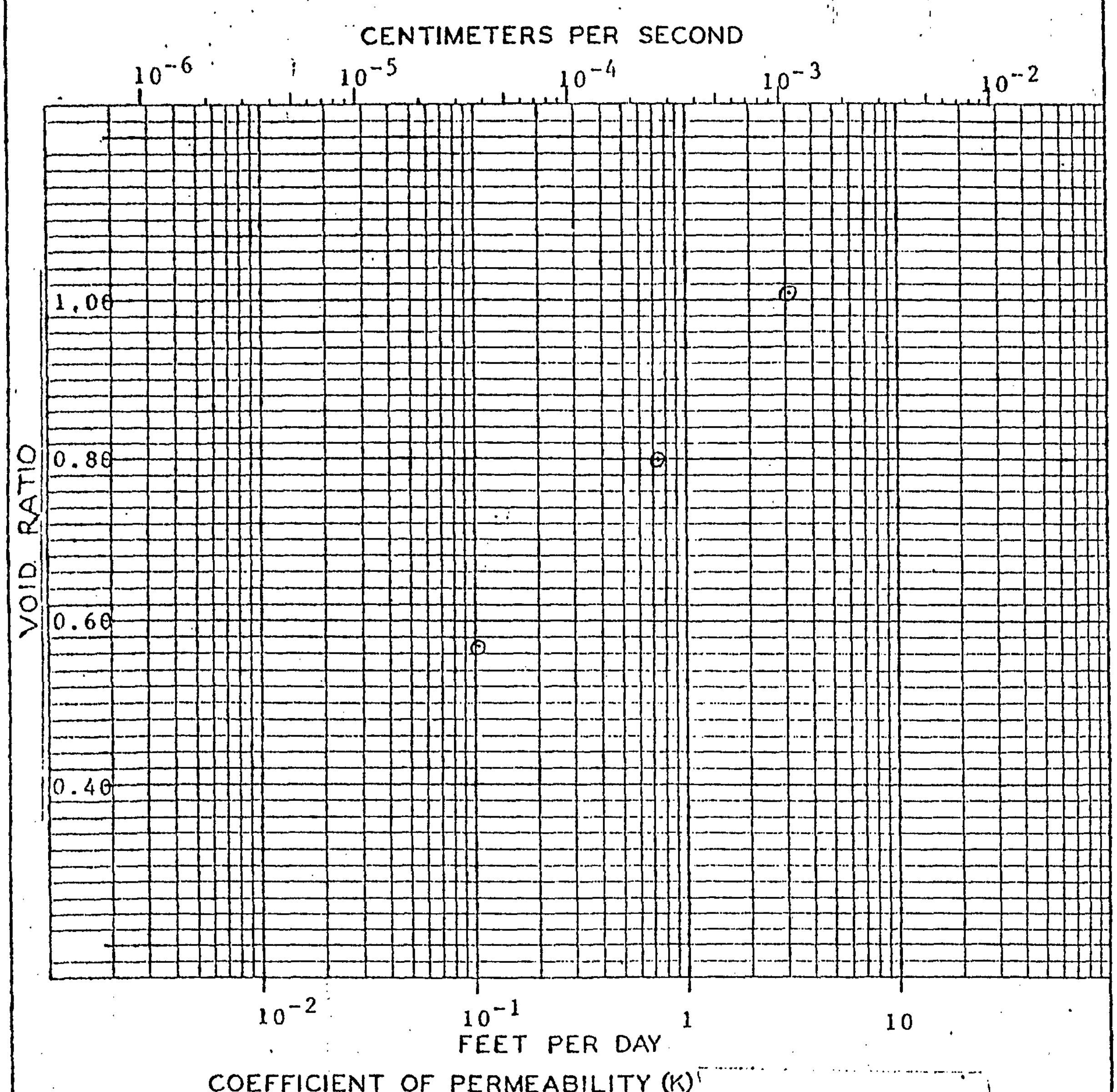
Div.No.	Hole No.	F.S.No.	Field Moisture Content (%)
103975	4F-87-1A	1A	2.9
103976	4F-87-1B	1B	6.6
103977	4F-87-2A	2A	8.4
103978	4F-87-2B	2B	6.1
103979	4F-87-3A	3A	5.4
103980	4F-87-3B	3B	6.6
103981	4F-87-4A	4A	6.3
103982	4F-87-4B	4B	8.6
103983	4F-87-5B	5B	9.5
103984	4F-87-6A	6A	3.5
103985	4F-87-6B	6B	4.2

PLATE 2A

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS: σ_3 = 0.5 tsf

CURVE	SPECIMEN DIAM. IN.	SPECIMEN HT. IN.	SPECIMEN MAX. PARTICLE	SPECIMEN CONDITION
	4.0	2.0	#4	REMOLDED TO 80 pcf 95 pcf 110 pcf

DISTRICT: SACRAMENTO

PROJECT: SAC. RIVER FLOOD CONTROL SYSTEM

CURVE	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
	COMP. A	1A, 1B 2A, 2B 3A, 3B 6A, 6B			

TESTED	COMPUTED	DRAWN	CHECKED

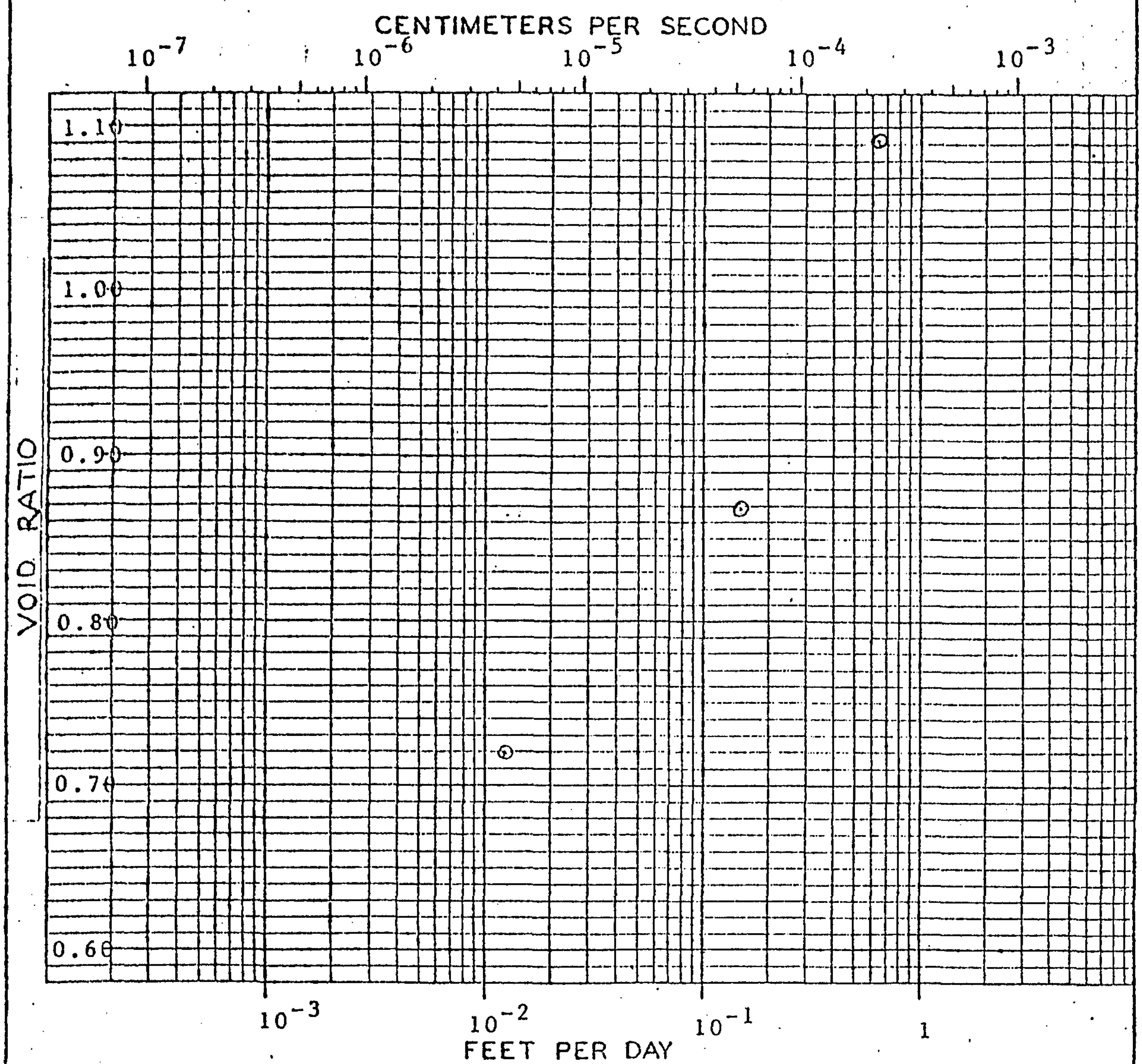
SPD FORM 67 (CIVIL) 6 OCT 55 REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 3

CORPS OF ENGINEERS U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



COEFFICIENT OF PERMEABILITY (K)

REMARKS: σ_3 =0.5 tsf

CURVE	SPECIMEN			
	DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
	4.0	2.0	#4	REMOLDED TO 80 pcf, 90 pcf & 100 pcf

DISTRICT: SACRAMENTO

PROJECT: SAC. RIVER FLOOD CONTROL SYSTEM

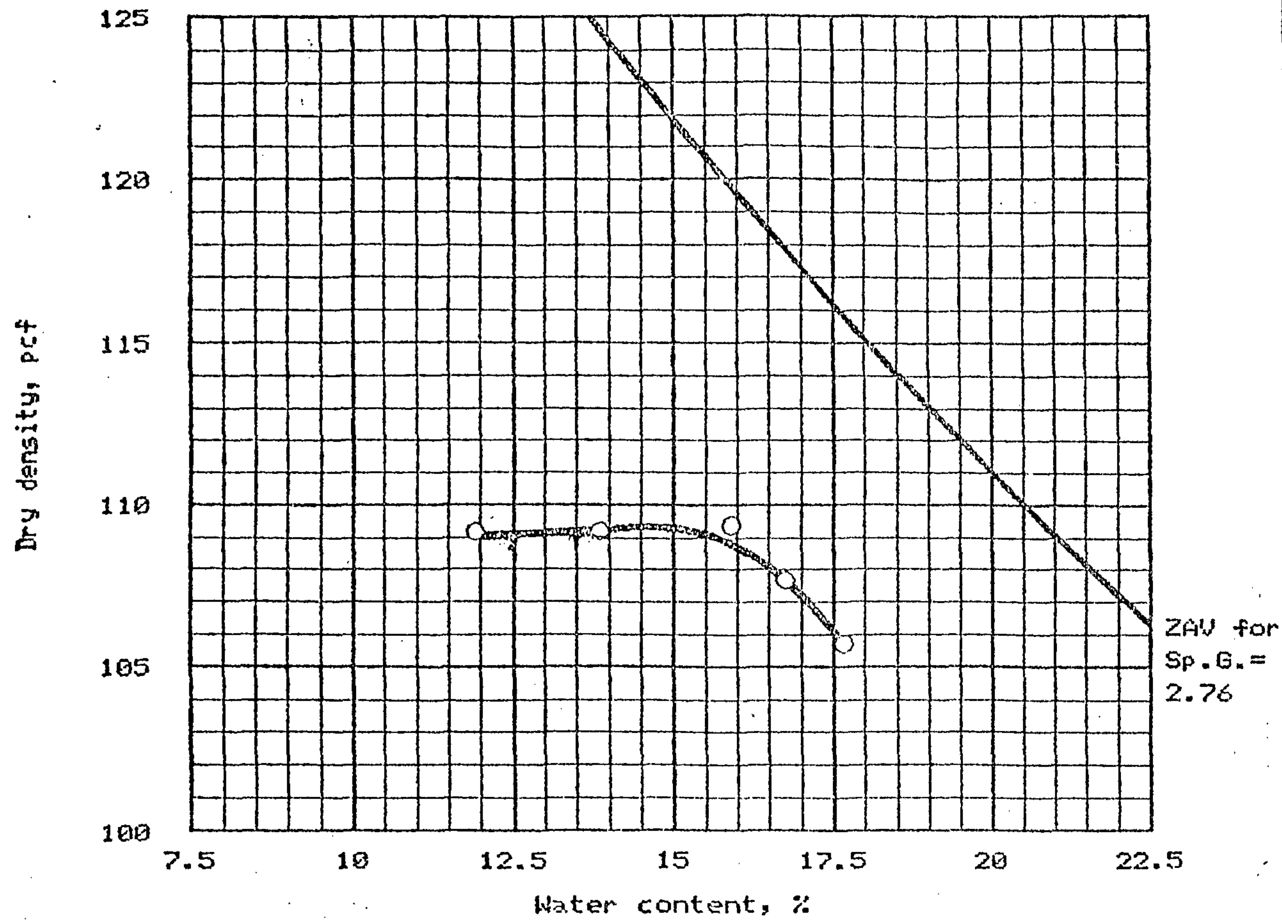
CURVE	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
	COMP. B	4A,4B & 5B			

TESTED	COMPUTED	DRAWN	CHECKED

SPD FORM 6 OCT 55 67 (CIVIL) REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 3A

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	Silty Sand (SM)	1.48 %	2.76		NP	0 %	31

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 14.7 % Maximum dry density = 109.4 pcf	Remolded

Division No.: 103964-69,73,74 HoleNo.4F-87-1A-3B-6A/-6B
Project: Sacramento River Flood Control(Amer.Riv.)
Location: Geotechnical Investigation.
F.S.No. 1A,3B,6A,6B
Date: 11-16-1987

Remarks:

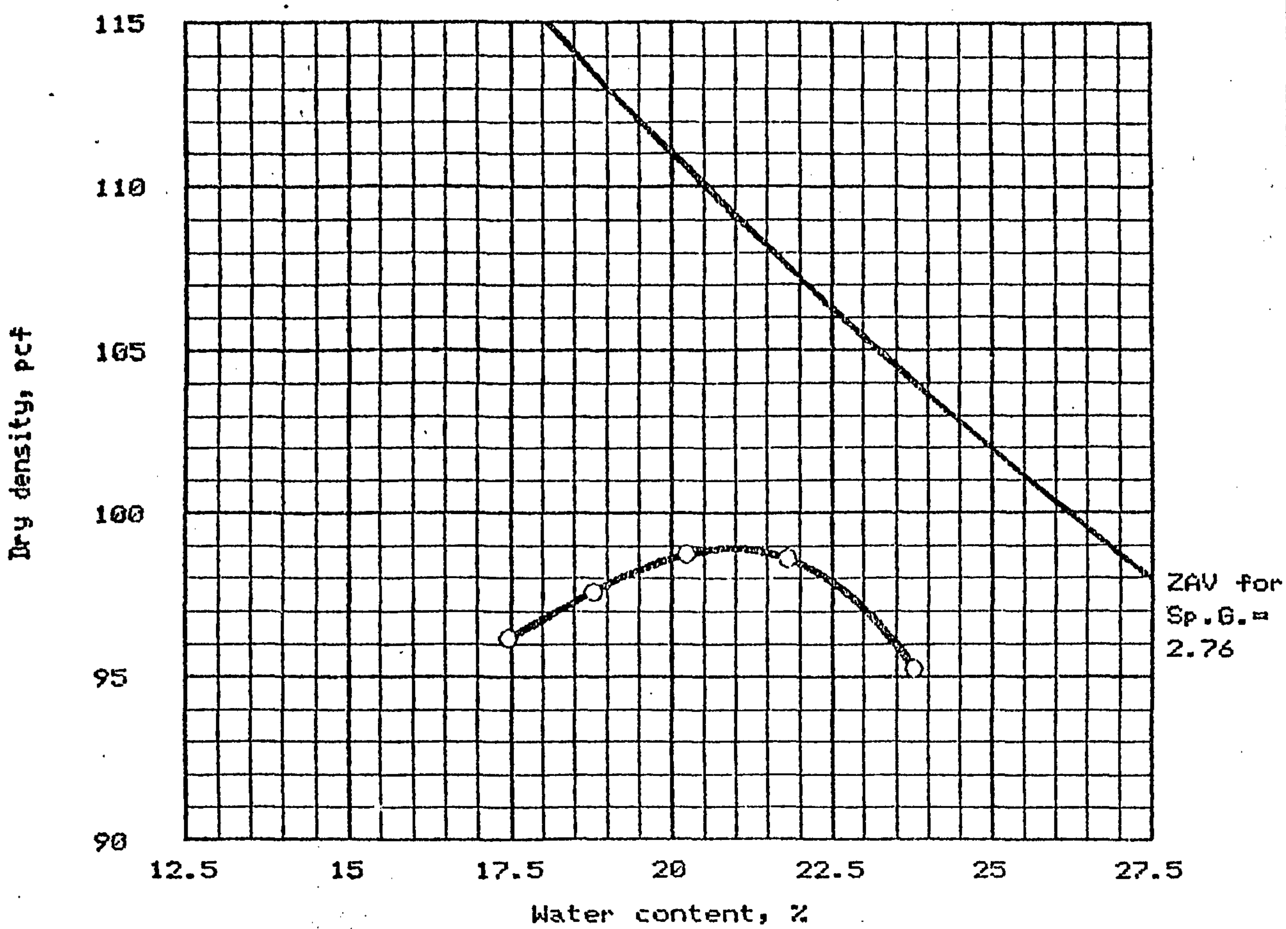
Composite A

COMPACTION TEST REPORT

CORPS OF ENGINEERS - SPDL

Plate No. 4

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	Sandy Silt(ML)	2.97 %	2.76	31	4	0 %	77

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 20.9 % Maximum dry density = 98.9 pcf	Remolded

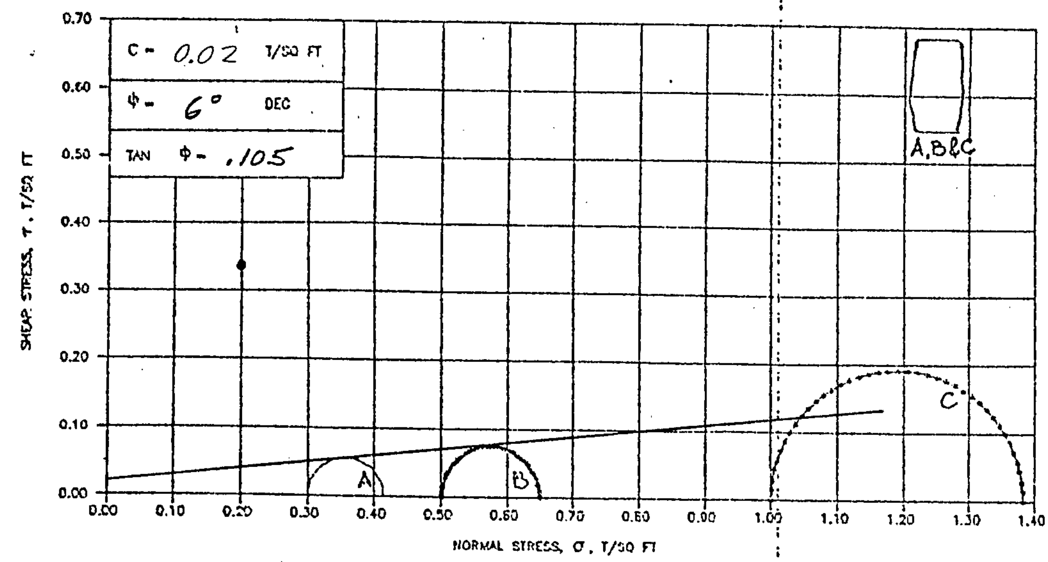
DivisionNo.: 103970,71,72 Hole No. 4F-87-4A-4B-5B
Project: Sacramento River Flood Control(Amer.Riv.)
Location: Geotechnical Investigation.
F.S.No. 4A, 4B, 5B
Date: 11-16-1987

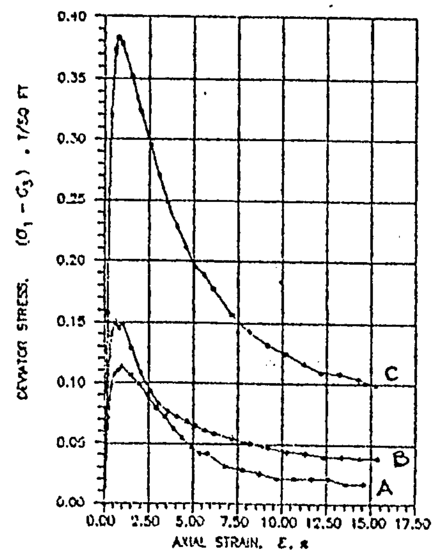
Remarks:
Composite B

COMPACTION TEST REPORT

CORPS OF ENGINEERS - SPDL

Plate No. 5





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	15.4	15.6	15.6
	DRY DENSITY LB/CU FT	γ_{d_o}	79.5	79.7	79.4
	SATURATION, %	S_o	37	37	37
	VOID RATIO	e_o	1.165	1.162	1.169
BEFORE SHEAR	WATER CONTENT, %	w_c	36.5	35.0	34.5
	DRY DENSITY LB/CU FT	γ_{d_c}	85.8	87.6	88.2
	SATURATION, %	S_c	100	100	100
	VOID RATIO	e_c	1.007	0.966	0.952
	FINAL BACK PRESSURE, T/SQ FT	u_b	6.5	7.2	6.5
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.3	0.5	1.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.11	0.15	0.38
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	13	15	19
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–
INITIAL DIAMETER, IN.		D_o	2.80	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.45

CONTROLLED STRAIN TEST

DESCRIPTION OF SPECIMEN Silty Sand(SM)

LL	PL NP	PI	G_s 2.76	TYPE OF SPECIMEN Remolded	TYPE OF TEST "$\overline{R}$"

REMARKS Remolded to 80 pcf at Opt water.
Max density - 109.4 pcf
Opt. water = 14.7%
Div. No. 103964,65,66,67,68,69, 103973 & 103974

PROJECT Sacramento River Flood Control System Evaluation (America River)

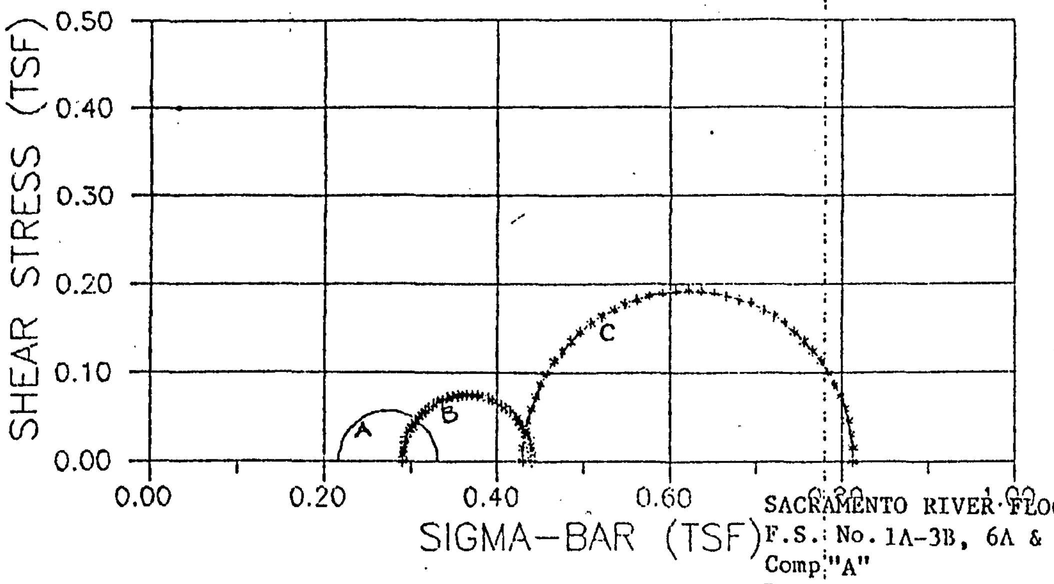
BORING NO. Comp "A" | SAMPLE NO. 1A-3 B, 6A &6B

LABORATORY SPDL | DATE March 1988

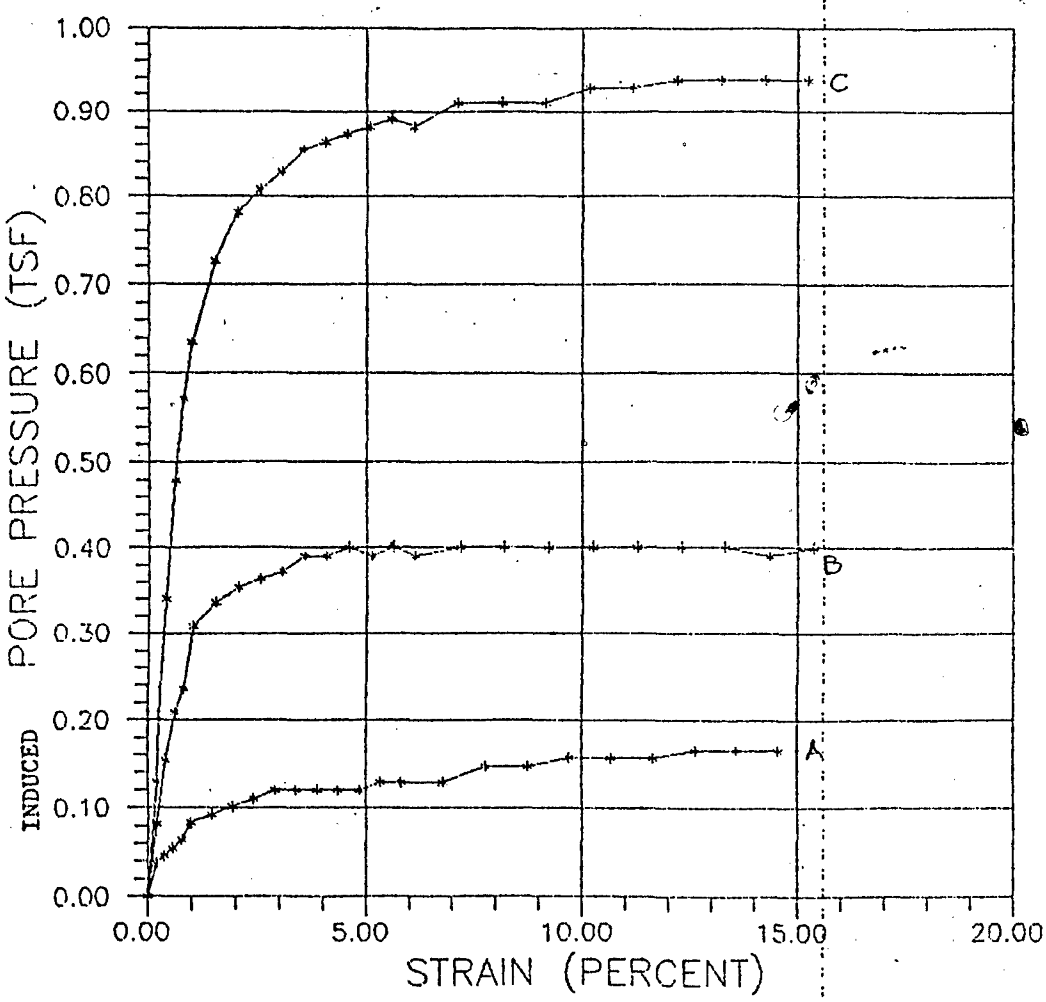
TRIAXIAL COMPRESSION TEST REPORT

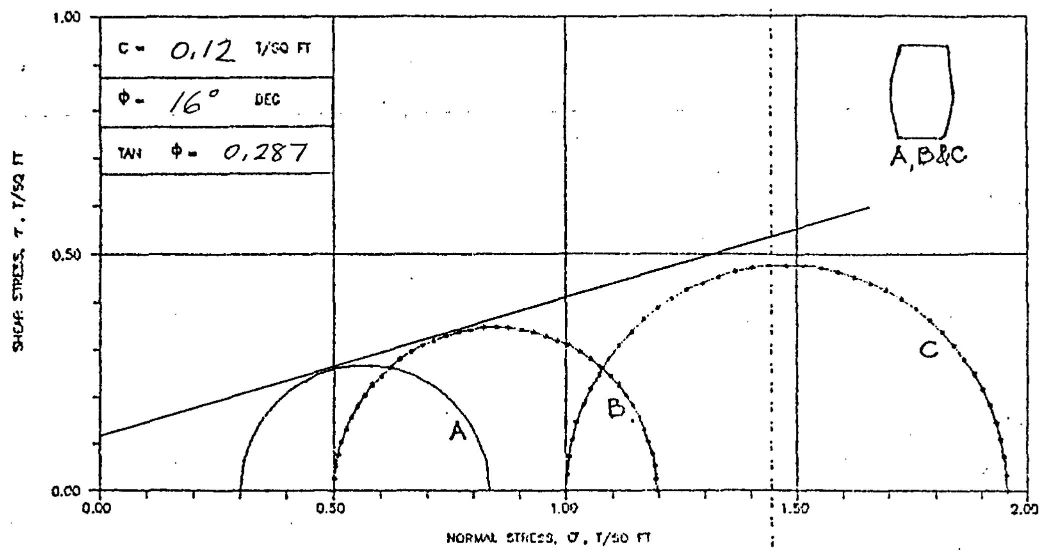
ENG FORM NO. 2083
REV. JUNE 1970

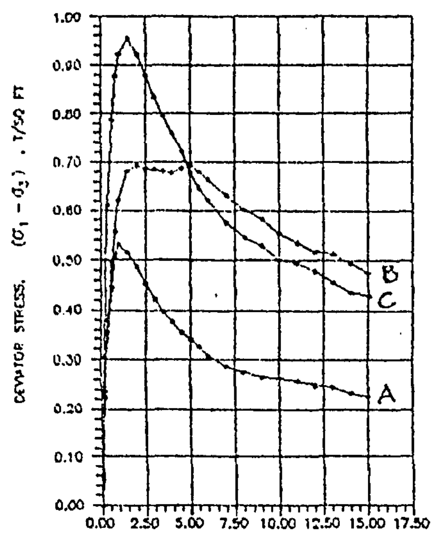
PLATE 6



SACRAMENTO RIVER FLOOD CONTROL
F.S. No. 1A-3B, 6A & 6B
Comp "A"
Div. No. 103964-69, 973 & 974







SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	14.8	14.8	14.7
	DRY DENSITY LB/CU FT	γ_{d_o}	94.9	95.0	95.0
	SATURATION, %	S_o	50	50	50
	VOID RATIO	e_o	0.814	0.813	0.813
BEFORE SHEAR	WATER CONTENT, %	w_c	27.7	27.0	26.7
	DRY DENSITY LB/CU FT	γ_{d_c}	97.6	98.7	100.0
	SATURATION, %	S_c	100	100	100
	VOID RATIO	e_c	0.764	0.745	0.738
	FINAL BACK PRESSURE, T/SQ FT	U_o	7.2	7.2	7.2
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.30	0.50	1.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.53	0.69	0.95
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	24	42	36
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	–	–	–
INITIAL DIAMETER, IN.		D_o	2.80	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.45

CONTROLLED STRAIN TEST

DESCRIPTION OF SPECIMEN Silty Sand (SM)

LL	PL NP	PI	G_s 2.76	TYPE OF SPECIMEN Remolded	TYPE OF TEST "$\overline{R}$"

REMARKS Remolded to 95 pcf at Opt water

Max. density = 109.4 pcf

Opt. water = 14.7%

Div. Nos. 103964-69 and 103973 and 103974.

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEMS EVALUATION (AMERICAN RIVER)

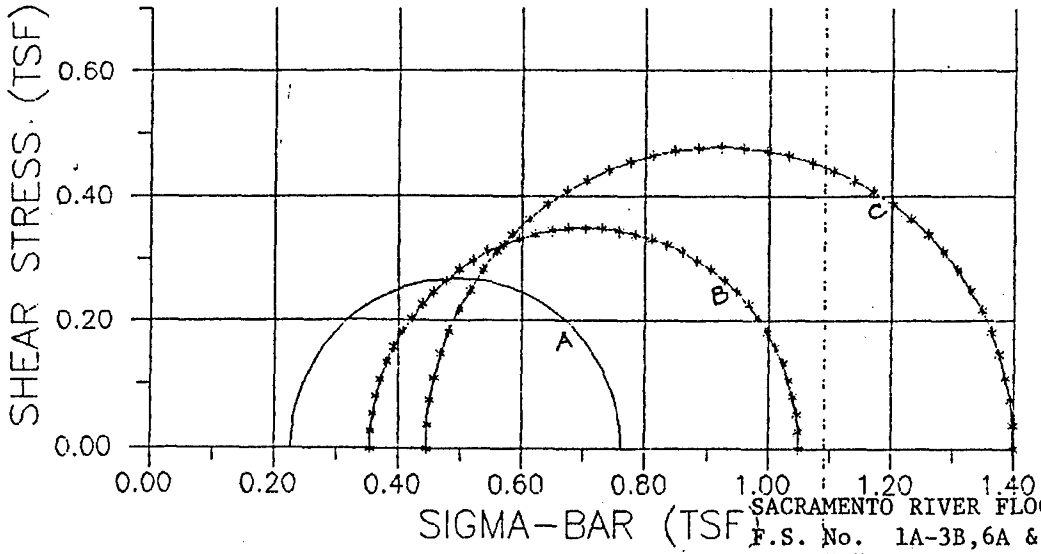
BORING NO. Comp "A" | SAMPLE NO. 1A-3B, 6A &6B

LABORATORY SPDL | DATE Mar 1988

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV. JUNE 1970

PLATE 8



SACRAMENTO RIVER FLOOD CONTROL
F.S. No. 1A-3B,6A & 6B
Comp "A"
Div Nos: 103964-69, 973 & 974

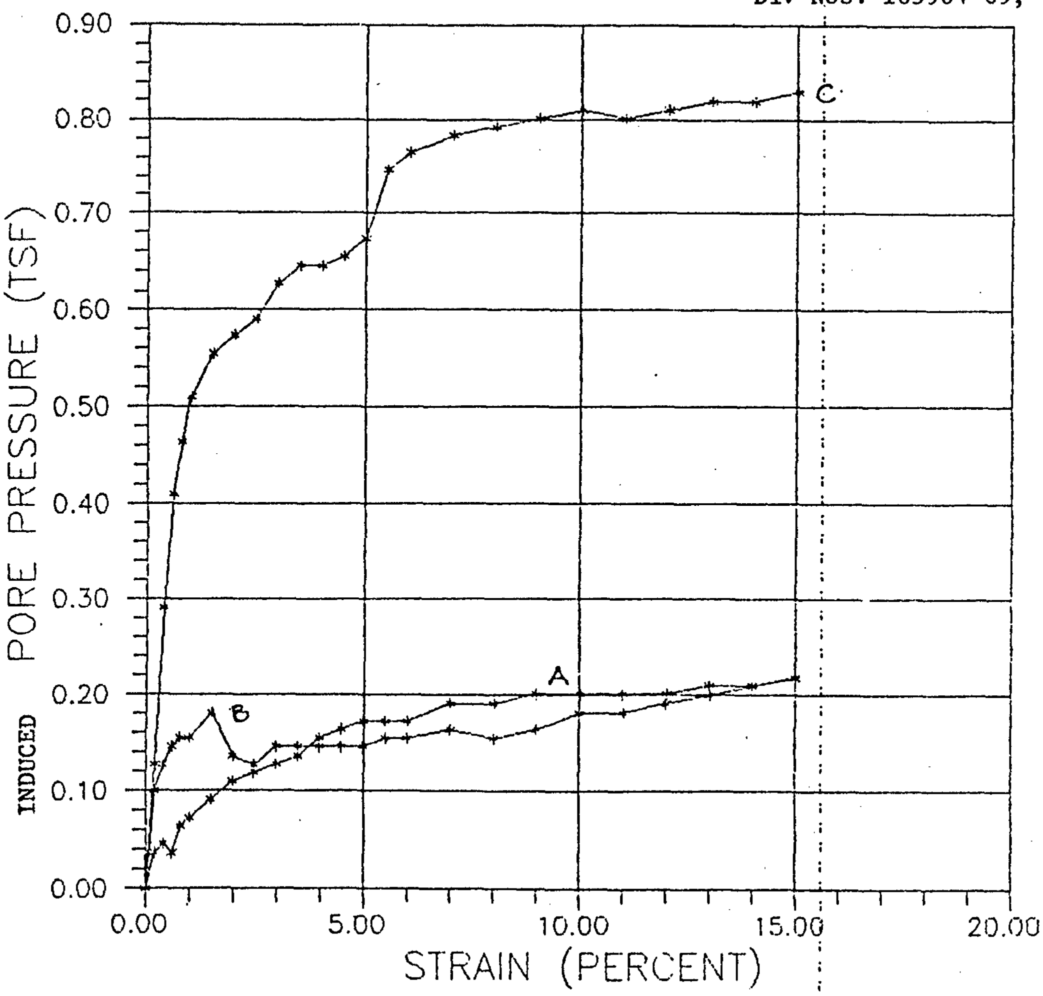
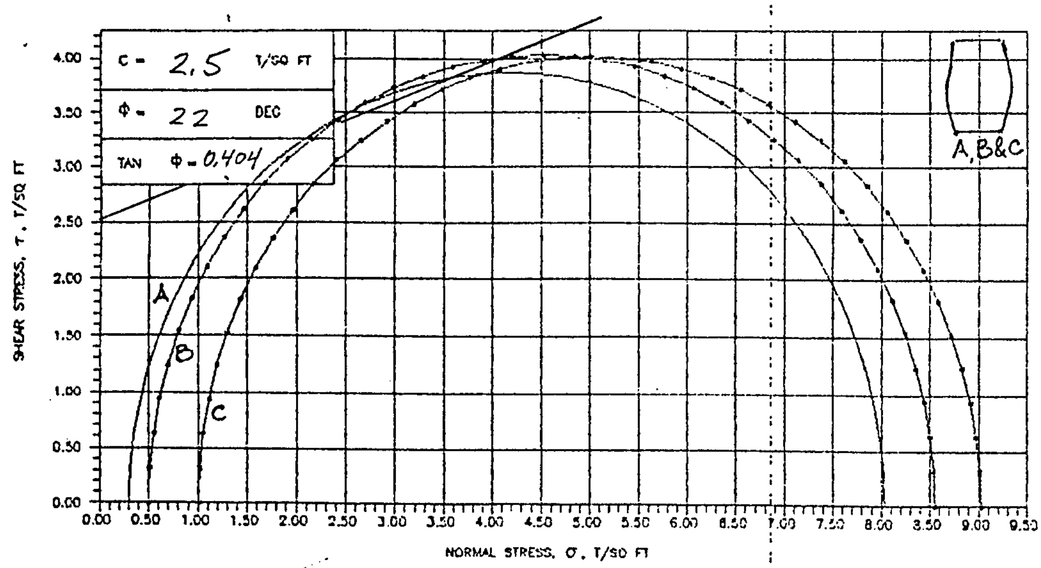
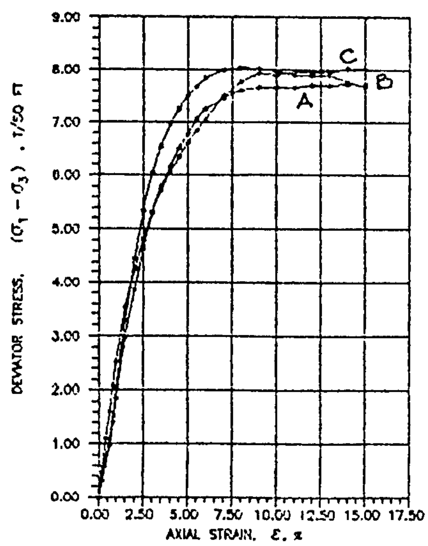


PLATE 9





	SPECIMEN NO.		A	B	C
INITIAL	WATER CONTENT, %	w_o	14.6	14.4	14.9
	DRY DENSITY LB/CU FT	γ_{d_o}	110.1	110.1	109.8
	SATURATION, %	S_o	71	71	72
	VOID RATIO	e_o	0.564	0.564	0.569
BEFORE SHEAR	WATER CONTENT, %	w_c	20.3	20.0	19.9
	DRY DENSITY LB/CU FT	γ_{d_c}	110.4	111.0	111.2
	SATURATION, %	S_c	100	100	100
	VOID RATIO	e_c	0.560	0.552	0.549
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.50	6.50	6.50
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	~~6.50~~ 0.30	~~6.50~~ 0.50	~~6.50~~ 1.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	7.73	8.05	8.02
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	342	133	309
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–
INITIAL DIAMETER, IN.		D_o	2.80	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.45

CONTROLLED STRAIN TEST

DESCRIPTION OF SPECIMEN Silty Sand(SM)

LL	PL NP	PI	G_s 2.76	TYPE OF SPECIMEN Remolded	TYPE OF TEST "$\overline{R}$"

REMARKS Remolded to 110 pcf at Opt. water

Max density = 109.4 pcf

Opt. water = 14.7%

Div. Nos. 103964, 65, 66, 67, 68

103969, 103973 & 103974

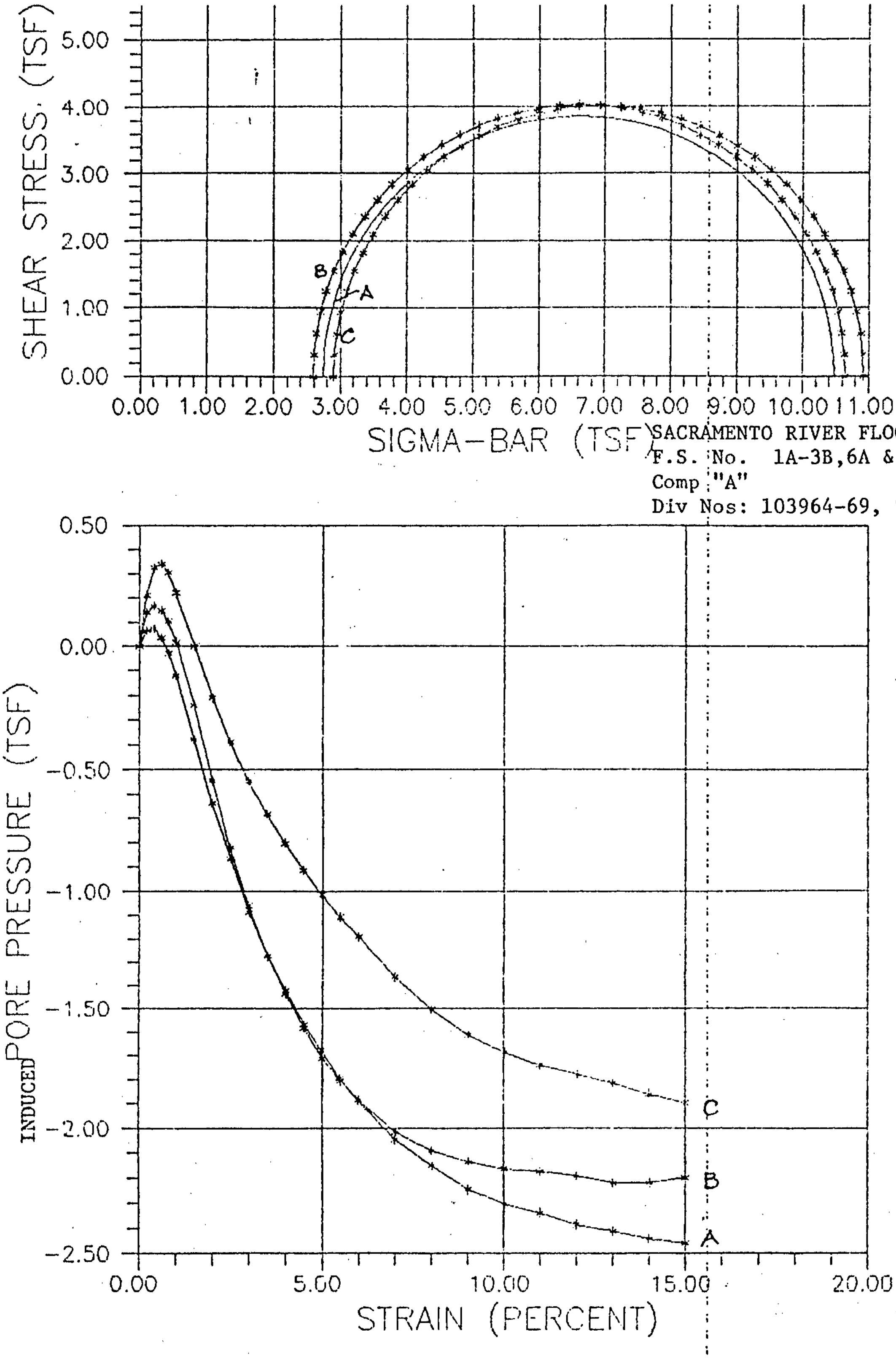
PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION (AMERICAN RIVER)

BORING NO Comp "A" | SAMPLE NO 1A-3B,6A & 6B

LABORATORY SPDL | DATE March 88

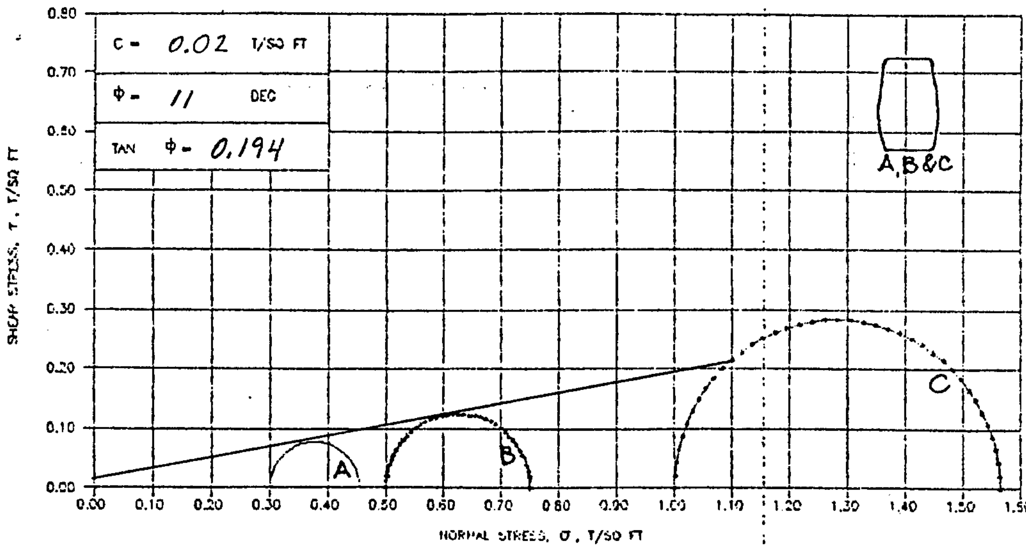
TRIAXIAL COMPRESSION TEST REPORT

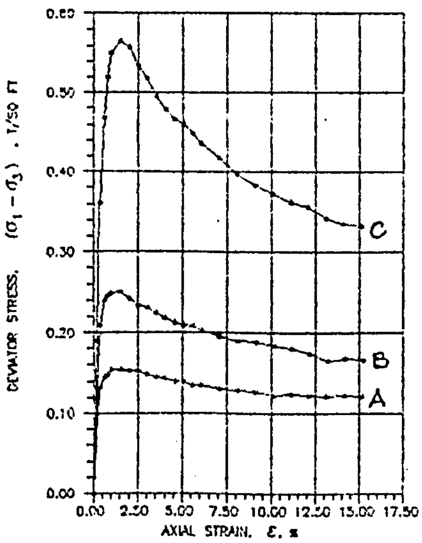
ENG FORM NO. 2089
REV. JUNE 1970



SACRAMENTO RIVER FLOOD CONTROL
F.S. No. 1A-3B,6A & 6B
Comp "A"
Div Nos: 103964-69, 973 & 974

PLATE 11





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	21.3	21.0	20.5
	DRY DENSITY LB/CU FT	γ_{d_o}	79.9	79.9	80.3
	SATURATION, %	S_o	51	50	50
	VOID RATIO	e_o	1.156	1.155	1.145
BEFORE SHEAR	WATER CONTENT, %	w_c	38.1	36.5	34.0
	DRY DENSITY LB/CU FT	γ_{d_c}	84.0	85.8	88.9
	SATURATION, %	S_c	100	100	100
	VOID RATIO	e_c	1.051	1.008	0.938
	FINAL BACK PRESSURE, T/SQ FT	U_o	7.2	7.2	7.2
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.3	0.5	1.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.15	0.25	0.56
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	22	16	31
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–
INITIAL DIAMETER, IN.		D_o	2.80	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.45

CONTROLLED STRAIN TEST

DESCRIPTION OF SPECIMEN Sandy Silt(ML)

LL 31	PL 27	PI 4	G_s 2.76	TYPE OF SPECIMEN Remolded	TYPE OF TEST "R"

REMARKS Remolded to 80 pcf at opt. water

Max. density = 98.9 pcf

Opt water = 20.9%

Div. No. 103970, 103971 & 103972

PROJECT Sacramento River Flood Control System Evaluation (Amercian River)

BORING NO. Comp "B"

SAMPLE NO 4A, 4B & 5B

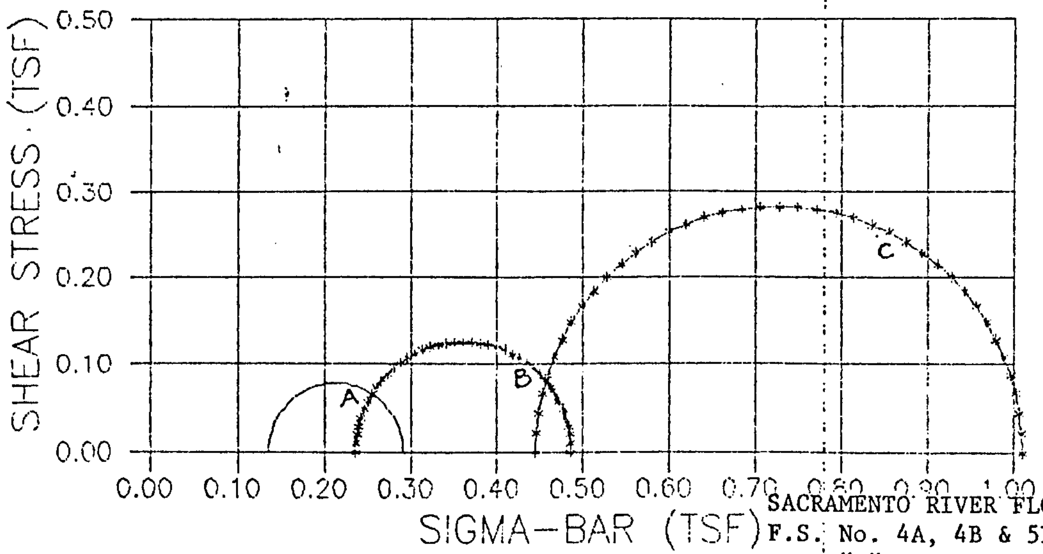
LABORATORY SPD

DATE March 1988

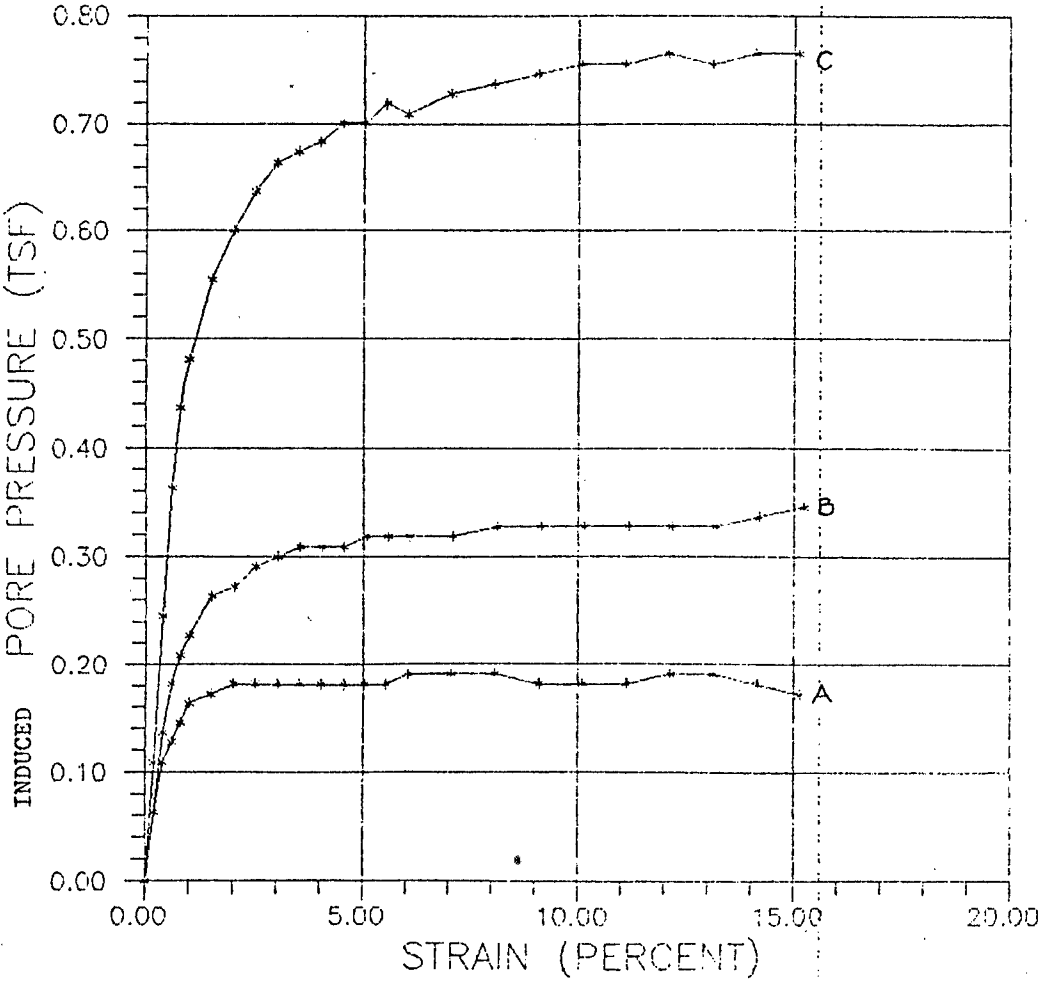
TRIAXIAL COMPRESSION TEST REPORT

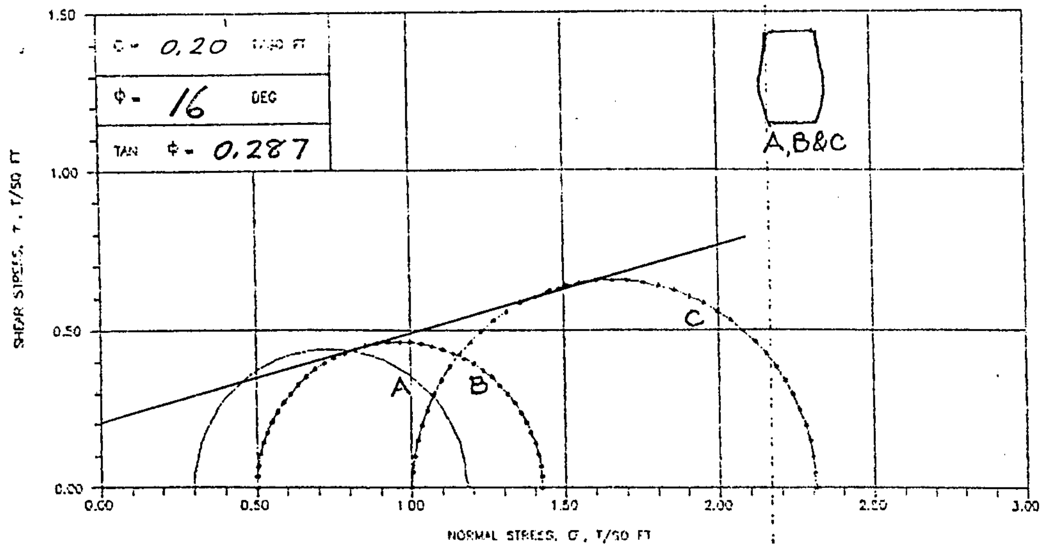
ENG FORM NO. 2089
REV. JUNE 1970

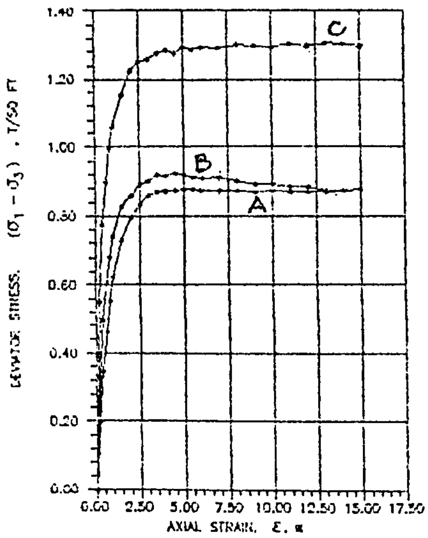
PLATE 12



SACRAMENTO RIVER FLOOD CONTROL
F.S. No. 4A, 4B & 5B
Comp "B"
Div. No. 103970, 103971 &
103972







		SPECIMEN NO.		A	B	C
INITIAL		WATER CONTENT, %	w_o	21.2	21.3	21.4
		DRY DENSITY LB/CU FT	γ_{d_o}	89.8	89.8	89.7
		SATURATION, %	S_o	64	64	64.1
		VOID RATIO	e_o	0.919	0.918	0.921
BEFORE SHEAR		WATER CONTENT, %	w_c	32.8	32.5	32.2
		DRY DENSITY LB/CU FT	γ_{d_c}	90.4	90.8	91.2
		SATURATION, %	S_c	100	100	100
		VOID RATIO	e_c	0.905	0.897	0.889
		FINAL BACK PRESSURE, T/SQ FT	u_o	7.2	7.2	7.2
MINOR PRINCIPAL STRESS, T/SQ FT			σ_3	0.3	0.5	1.0
MAXIMUM DEVIATOR STRESS, T/SQ FT			$(\sigma_1 - \sigma_3)_{MAX}$	0.88	0.92	1.31
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN			t_f	97	66	245
ULTIMATE DEVIATOR STRESS, T/SQ FT			$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–
INITIAL DIAMETER, IN.			D_o	2.80	2.80	2.80
INITIAL HEIGHT, IN.			H_o	6.45	6.45	6.45

CONTROLLED STRAIN TEST

DESCRIPTION OF SPECIMEN Sandy Silt(ML)

LL 31 | PL 27 | PI 4 | G_s 2.76 | TYPE OF SPECIMEN Remolded | TYPE OF TEST "$\bar{R}$"

REMARKS Remolded to 90pcf at Opt. water
Max. Density =98.9 pcf
Opt. water = 20.9%
Div. No. 103970, 103971 and 103972

PROJECT Sacramento River Flood Control System Evaluation (American River)

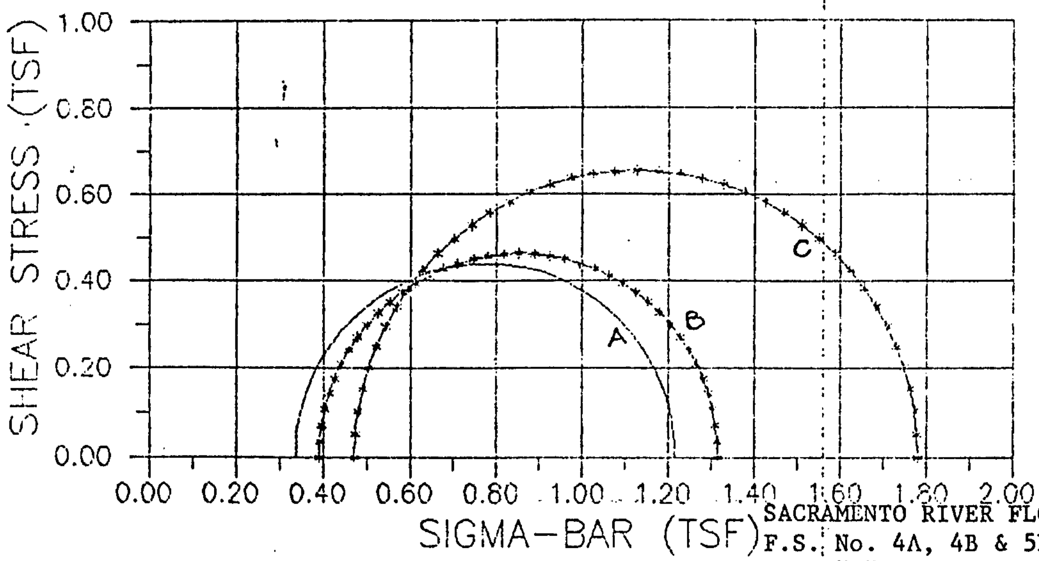
BORING NO. Comp "B" SAMPLE NO. 4A, 4B & 5B

LABORATORY SPDL DATE March 1988

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV. JUNE 1970

PLATE 14



SACRAMENTO RIVER FLOOD CONTROL
F.S. No. 4A, 4B & 5B
Comp "B"
Div. No. 103970.103971 &
103972

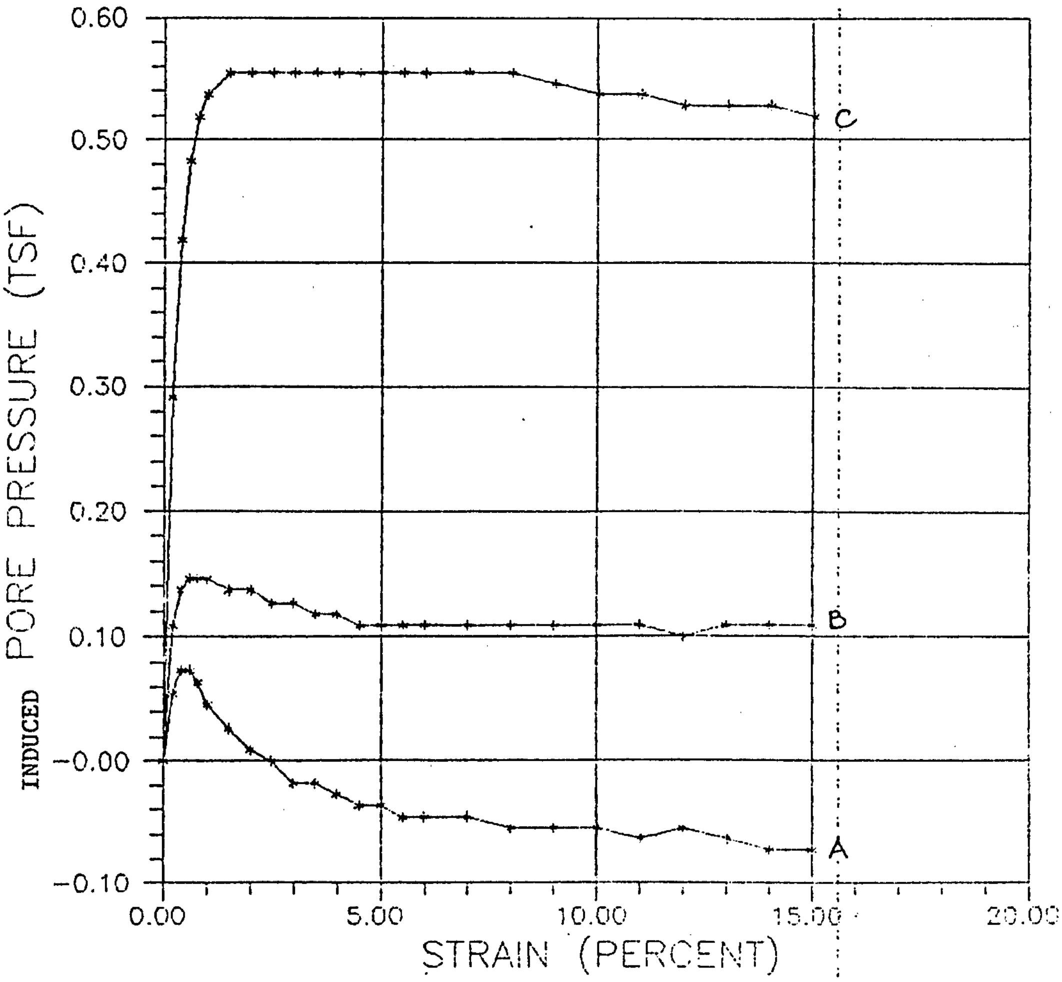
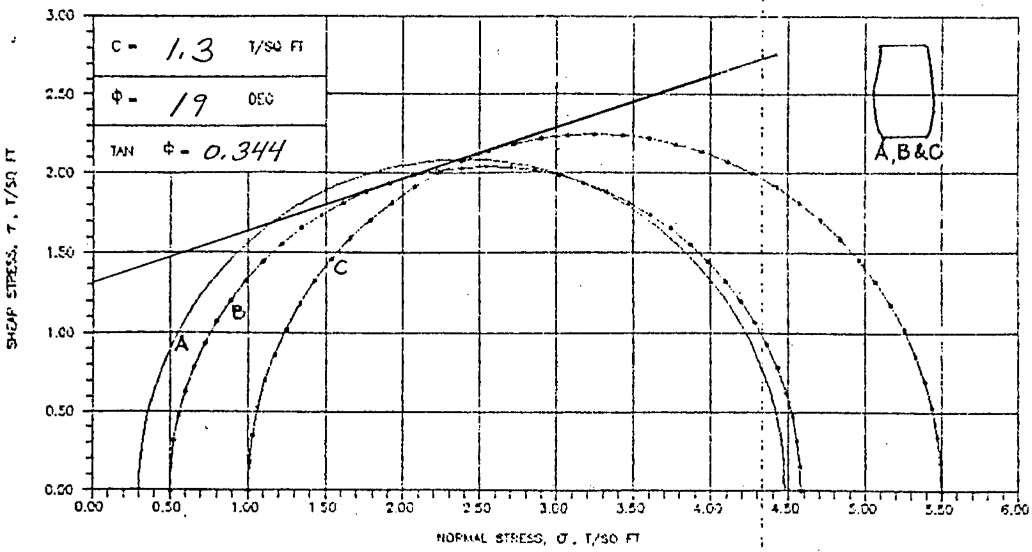
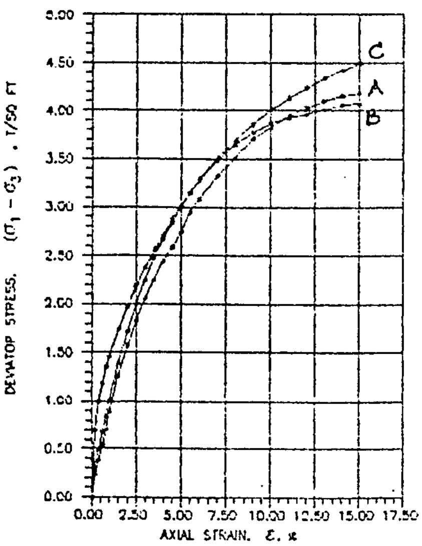


PLATE 15





			SPECIMEN NO.	A	B	C
INITIAL	WATER CONTENT, %		w_o	20.9	20.9	20.3
	DRY DENSITY LB/CU FT		γ_{d_o}	99.8	99.9	100.3
	SATURATION, %		S_o	79	79	78
	VOID RATIO		e_o	0.726	0.724	0.717
BEFORE SHEAR	WATER CONTENT, %		w_c	26.2	26.0	25.7
	DRY DENSITY LB/CU FT		γ_{d_c}	99.9	100.3	100.8
	SATURATION, %		S_c	100	100	100
	VOID RATIO		e_c	0.723	0.718	0.709
	FINAL BACK PRESSURE, T/SQ FT		U_c	7.2	7.2	7.2
MINOR PRINCIPAL STRESS, T/SQ FT			σ_3	0.3	0.5	1.0
MAXIMUM DEVIATOR STRESS, T/SQ FT			$(\sigma_1 - \sigma_3)_{MAX}$	4.18	4.08	4.50
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN			t_f	266	360	275
ULTIMATE DEVIATOR STRESS, T/SQ FT			$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–
INITIAL DIAMETER, IN			D_o	2.80	2.80	2.80
INITIAL HEIGHT, IN.			H_o	6.45	6.45	6.45

CONTROLLED STRAIN TEST

DESCRIPTION OF SPECIMEN Sandy Silt (ML)

LL 31	PL 27	PI 4	G_S 2.76	TYPE OF SPECIMEN Remolded	TYPE OF TEST "$\bar{R}$"

REMARKS Remolded to 100 pcf at Opt. water.

Max density - 98.9 pcf

Opt. water = 20.9%

Div. No. 103970, 103971 and 103972

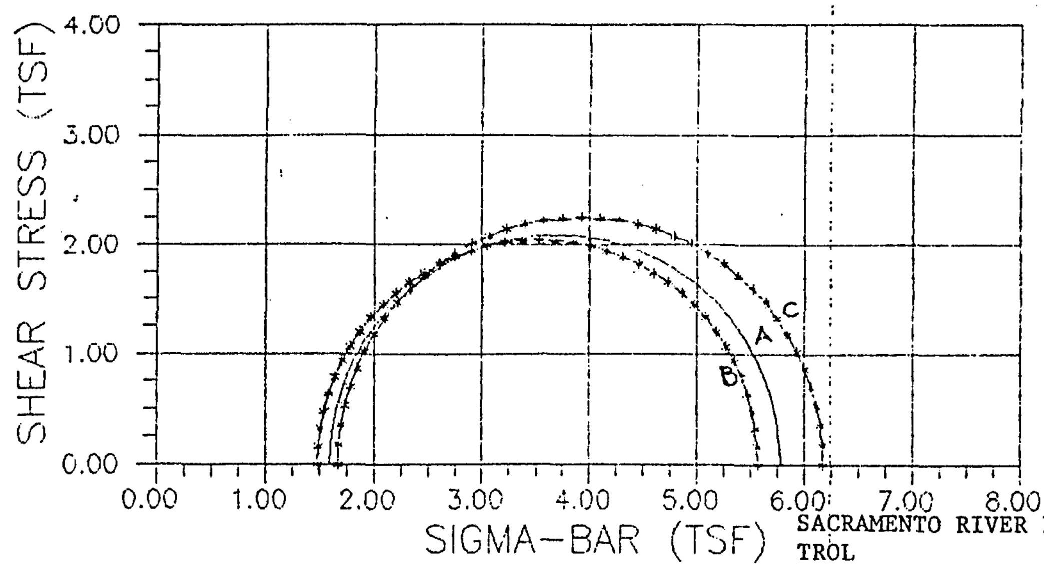
PROJECT Sacramento River Flood Control System Evaluation (American River)

BORING NO. Comp "B" | SAMPLE NO. 4A, 4B & 5B

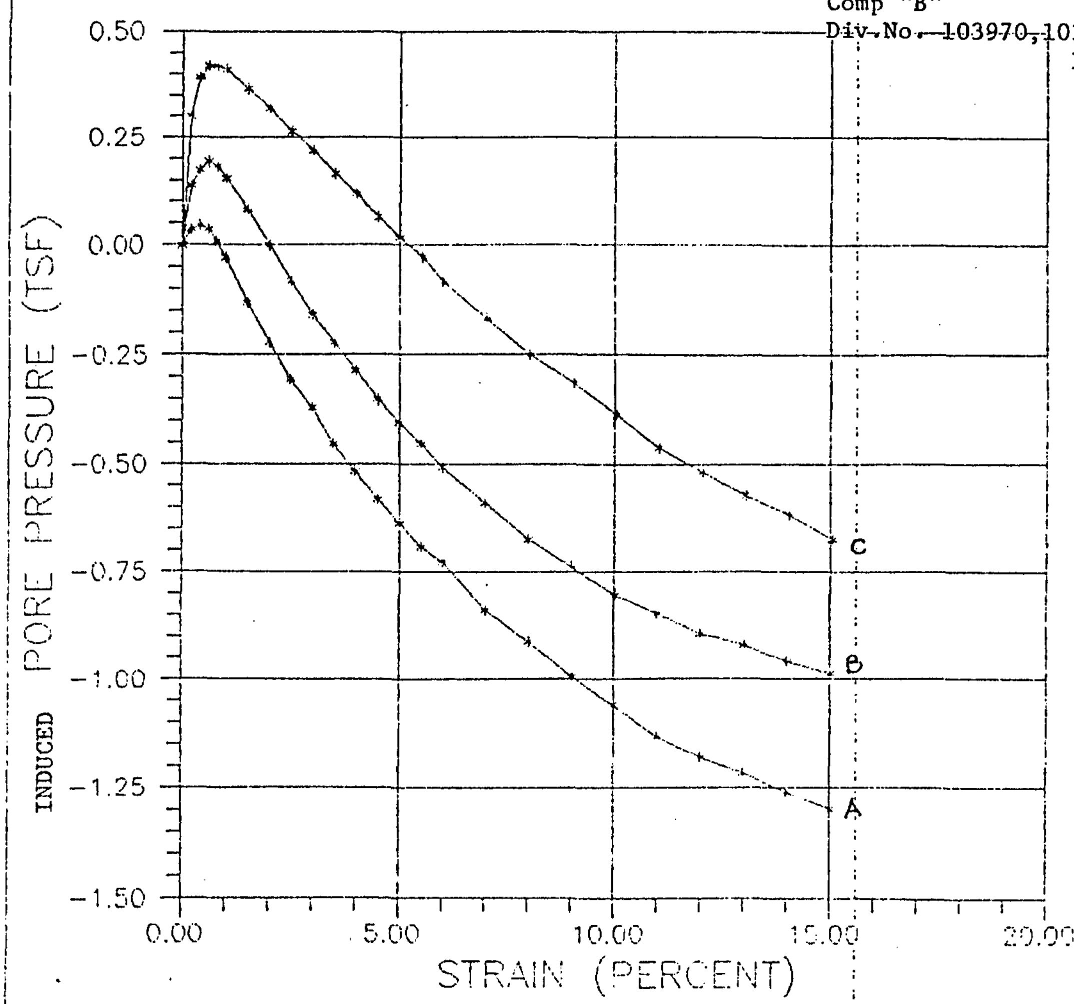
LABORATORY SPDL | DATE March 1988

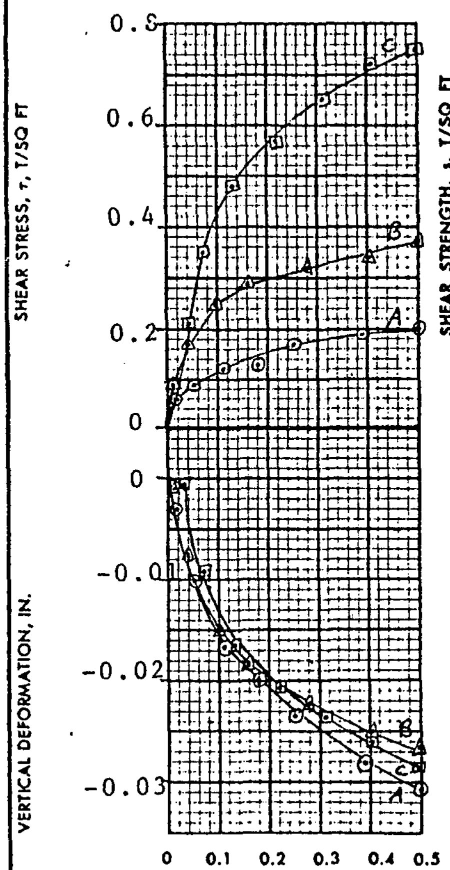
TRIAXIAL COMPRESSION TEST REPORT

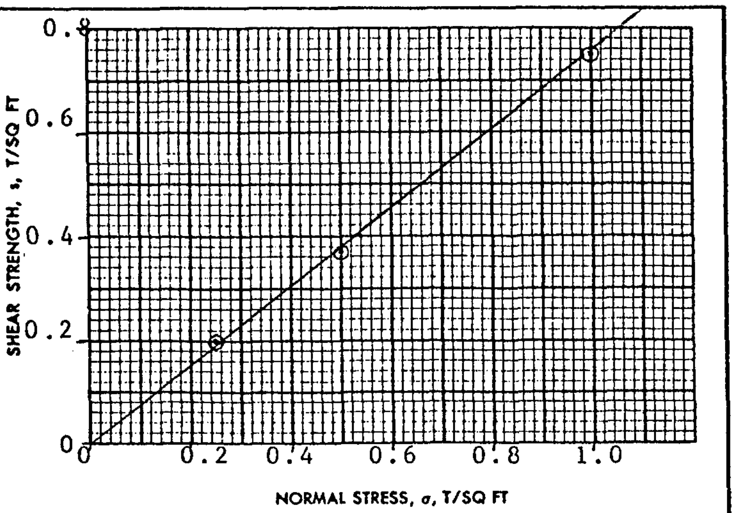
ENG FORM NO. 2089
REV. JUNE 1970



SACRAMENTO RIVER FLOOD CONTROL
F.S. No. 4a, 4B, & 5B
Comp "B"
~~Div. No. 103970,~~ 103971, &
103972







SHEAR STRENGTH PARAMETERS

ϕ' = 37.0

TAN ϕ' = 0.753

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			A	B	C	
INITIAL	WATER CONTENT	w_o	15.4%	14.7%	15.2%	%
	VOID RATIO	e_o	1.17	1.15	1.16	
	SATURATION	S_o	37 %	35 %	36 %	%
	DRY DENSITY, LB/CU FT	γ_d	79.5	80.0	79.7	
VOID RATIO AFTER CONSOLIDATION		e_c	.990	.980	.921	
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	–	–	–	
FINAL	WATER CONTENT	w_f	26.6%	27.0%	26 0%	%
	VOID RATIO	e_f	.857	.864	.794	
	SATURATION	S_f	86 %	87%	90 %	%
NORMAL STRESS, T/SQ FT		σ	0.25	0.50	1.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.20	0.37	0.75	
ACTUAL TIME TO FAILURE, MIN		t_f	298	353	353	
RATE OF STRAIN, IN./MIN			.0017	.0014	.0014	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	–	–	–	

TYPE OF SPECIMEN Remolded	3.25 IN. SQUARE	2.76 IN. THICK	
CLASSIFICATION Silty Sand (SM)			
LL	PL	PI NP	G_s 2.76

REMARKS Div. No. "Composite A" (1A, 1B, 2A, 2B, 3A, 3B, 6A &6B)

Dry density=80pcf

PROJECT Sacramento River Flood Control System Evaluation (American River)

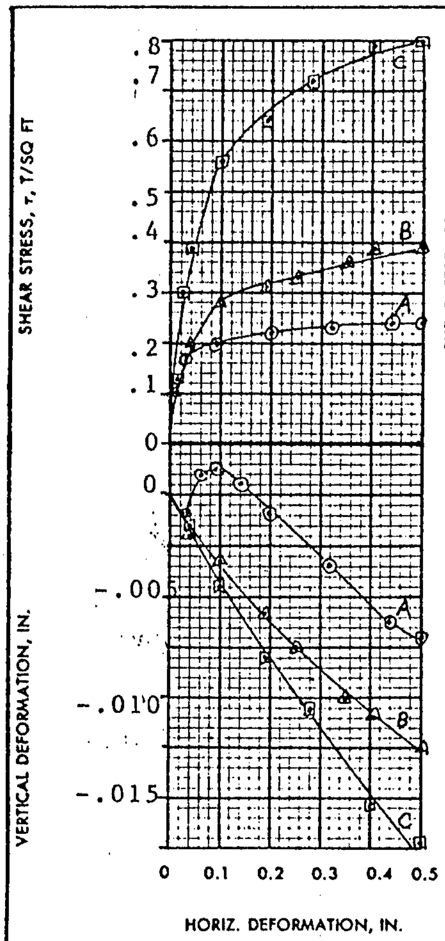
AREA

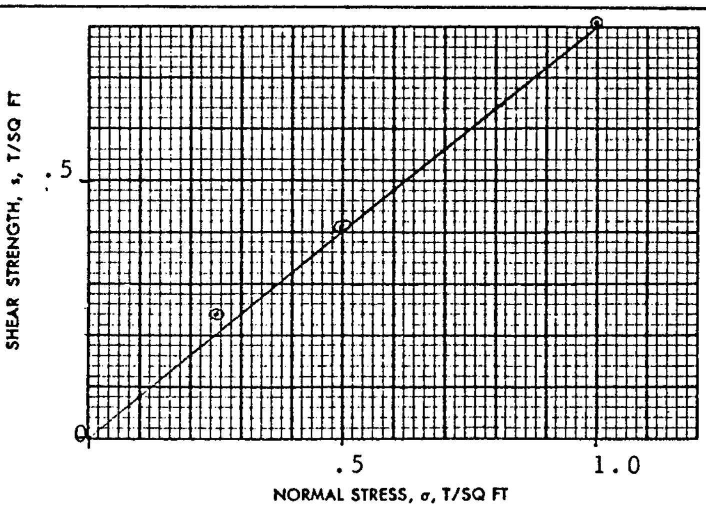
BORING NO. 4F-87 | SAMPLE NO.

DEPTH EL | DATE January 1988

DIRECT SHEAR TEST REPORT

PLATE 18





TEST NO.			A	B	C	
INITIAL	WATER CONTENT	w_o	14.6 %	14.6 %	14.6 %	%
	VOID RATIO	e_o	.814	.814	.813	
	SATURATION	S_o	50 %	49 %	50 %	%
	DRY DENSITY, LB/CU FT	γ_d	94.9	95.0	95.0	
VOID RATIO AFTER CONSOLIDATION		e_c	.805	.796	.791	
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	–	–	–	
FINAL	WATER CONTENT	w_f	25.0 %	26.3 %	25.1 %	%
	VOID RATIO	e_f	.777	.749	.727	
	SATURATION	S_f	89 %	97 %	95 %	%
NORMAL STRESS, T/SQ FT		σ	0.25	0.50	0.10	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.24	0.41	0.81	
ACTUAL TIME TO FAILURE, MIN		t_f	330	330	330	
RATE OF STRAIN, IN./MIN			.0014	.0015	.0015	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	–	–	–	

SHEAR STRENGTH PARAMETERS

ϕ' = 39.0

TAN ϕ' = 0.810

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN Remolded		3.25 IN. SQUARE	0.5 IN. THICK	
CLASSIFICATION Silty Sand (SM)				
LL	PL	PI NP		G_s 2.76

REMARKS Div.No."Composite A" (1A,1B,2A,2B,3A,3B,6A &6B)

Dry density = 95pcf

PROJECT Sacramento River Flood Control System Evaluation (American River)

AREA

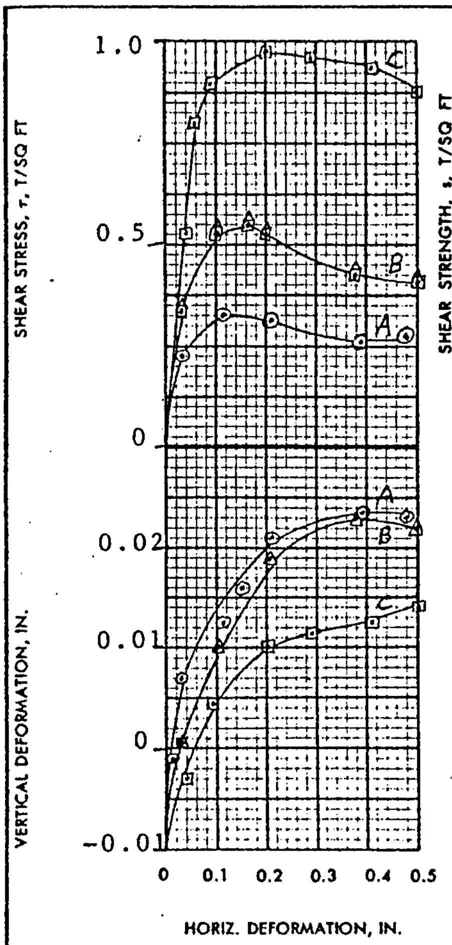
BORING NO. 4F-87. SAMPLE NO. —

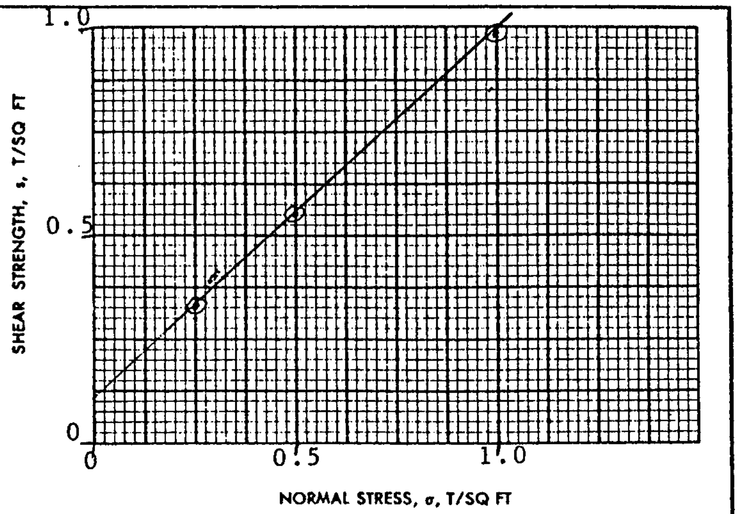
DEPTH EL

DATE January 1988

DIRECT SHEAR TEST REPORT

PLATE 19





TEST NO.			A	B	C	
INITIAL	WATER CONTENT	w_o	15.6%	15.3%	15.6%	%
	VOID RATIO	e_o	.583	.579	.579	
	SATURATION	S_o	74 %	73 %	74 %	%
	DRY DENSITY, LB/CU FT	γ_d	108.8	109.1	109.1	
VOID RATIO AFTER CONSOLIDATION		e_c	.573	.561	.554	
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	–	–	–	
FINAL	WATER CONTENT	w_f	21.9%	21.4%	21.4%	%
	VOID RATIO	e_f	.627	.631	.598	
	SATURATION	S_f	97 %	97 %	99 %	%
NORMAL STRESS, T/SQ FT		σ	0.25	0.50	1.00	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.33	0.55	0.97	
ACTUAL TIME TO FAILURE, MIN		t_f	355	293	360	
RATE OF STRAIN, IN./MIN			.0014	.0017	.0014	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	–	–	–	

SHEAR STRENGTH PARAMETERS

ϕ' = 41.0

TAN ϕ' = 0.869

c' = 0.10 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN Remolded — 3.25 IN. SQUARE — 0.5 IN. THICK

CLASSIFICATION Silty Sand (SM)

LL | PL | PI NP | G. 2.76

REMARKS Div.No."Composite A" (1A,1B,2A,2B,3A,3B,6A &6B)

Dry density = 110pcf

PROJECT Sacramento River Flood System Evaluation (American River)

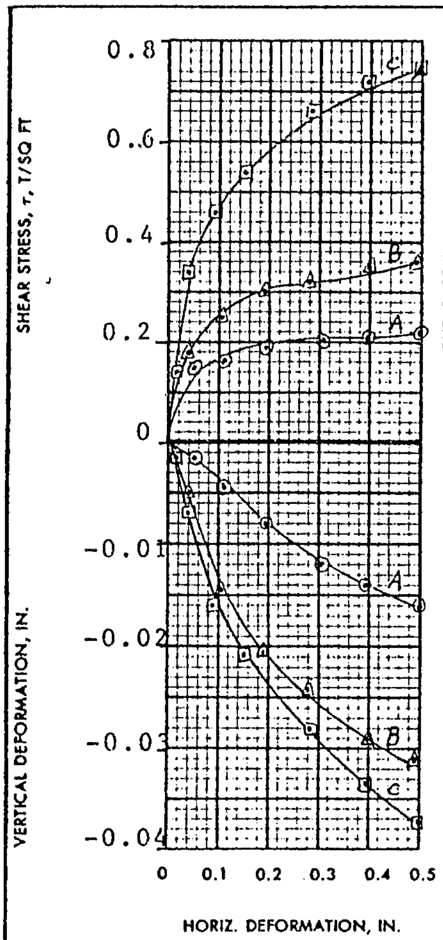
AREA

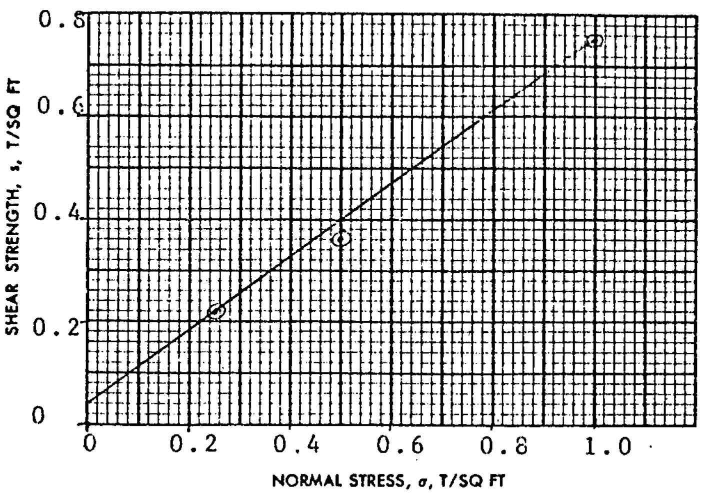
BORING NO. 4F-87 | SAMPLE NO.

DEPTH EL | DATE January 1988

DIRECT SHEAR TEST REPORT

PLATE 20





SHEAR STRENGTH PARAMETERS

ϕ' = 35.5

TAN ϕ' = 0.713

c' = 0.04 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			A	B	C	
INITIAL	WATER CONTENT	w_o	20.1%	20.9%	20.5 %	%
	VOID RATIO	e_o	1.140	1.153	1.150	
	SATURATION	S_o	49 %	50 %	49 %	%
	DRY DENSITY, LB/CU FT	γ_d	80.5	80.0	80.1	
VOID RATIO AFTER CONSOLIDATION		e_c	1.125	1.091	1.058	
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	–	–	–	
FINAL	WATER CONTENT	w_f	34.9%	33.7%	29.9%	%
	VOID RATIO	e_f	1.031	.931	.877	
	SATURATION	S_f	93 %	100 %	94 %	%
NORMAL STRESS, T/SQ FT		σ	0.25	0.50	1.00	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.22	0.36	0.75	
ACTUAL TIME TO FAILURE, MIN		t_f	354	342	372	
RATE OF STRAIN, IN./MIN			.0014	.0015	.0013	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	–	–	–	

TYPE OF SPECIMEN Remolded	3.25 IN. SQUARE	0.5 IN. THICK	
CLASSIFICATION Sandy Silt (ML)			
LL 31	PL 27	PI 4	G_s 2.76

REMARKS Div.No."Composite B" (4A, 4B &5B)

Dry density = 80pcf

PROJECT Sacramento River Flood Control System Evaluation (American River)

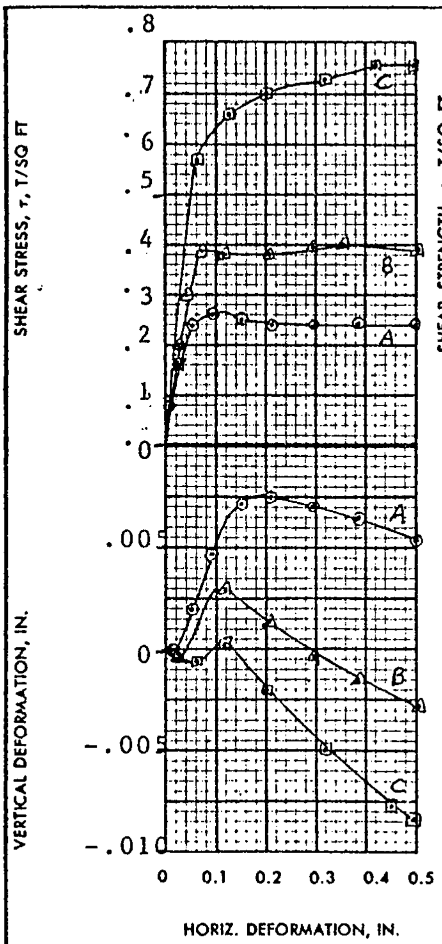
AREA

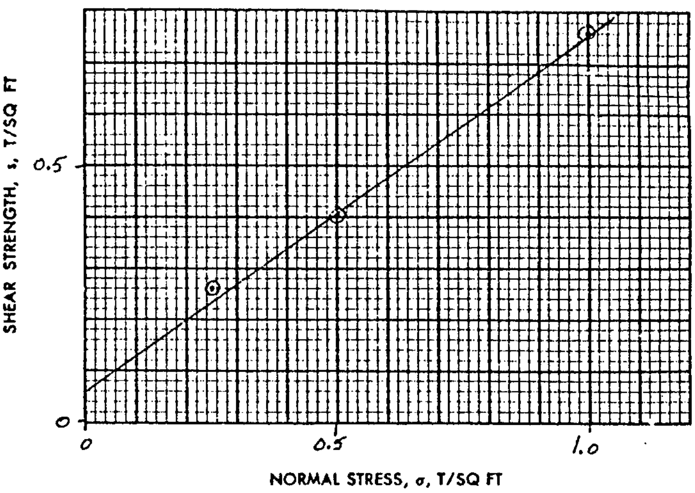
BORING NO. 4F-87 | SAMPLE NO.

DEPTH EL | DATE January 1988

DIRECT SHEAR TEST REPORT

PLATE 21





SHEAR STRENGTH PARAMETERS

φ' = 36.5

TAN φ' = 0.740

c' = 0.06 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			A	B	C	
INITIAL	WATER CONTENT	w_o	20.7%	20.9%	20.7%	%
	VOID RATIO	e_o	.913	.912	.911	
	SATURATION	S_o	63 %	63 %	63 %	%
	DRY DENSITY, LB/CU FT	γ_d	90.0	90.1	90.1	
VOID RATIO AFTER CONSOLIDATION		e_c	.906	.889	.871	
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	–	–	–	
FINAL	WATER CONTENT	w_f	33.3%	31.8%	30.1%	%
	VOID RATIO	e_f	.926	.878	.837	
	SATURATION	S_f	99 %	100 %	99 %	%
NORMAL STRESS, T/SQ FT		σ	0.25	0.50	1.00	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.26	0.40	0.76	
ACTUAL TIME TO FAILURE, MIN		t_f	350	345	350	
RATE OF STRAIN, IN./MIN			.0014	.0015	.0014	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	–	–	–	

TYPE OF SPECIMEN Remolded — 3.25 IN. SQUARE — 0.5 IN. THICK

CLASSIFICATION Sandy Silt (ML)

LL 31 | PL 27 | PI 4 | G_s 2.76

REMARKS Div.No."CompositeB" (4A,4B&5B)

Dry density = 90pcf

PROJECT Sacramento River Flood Control System Evaluation (American River)

AREA

BORING NO. 4F-87 SAMPLE NO.

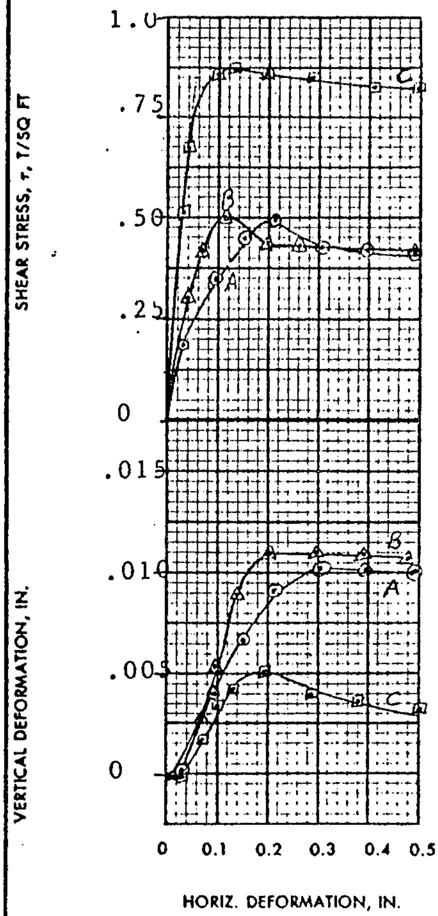
DEPTH EL

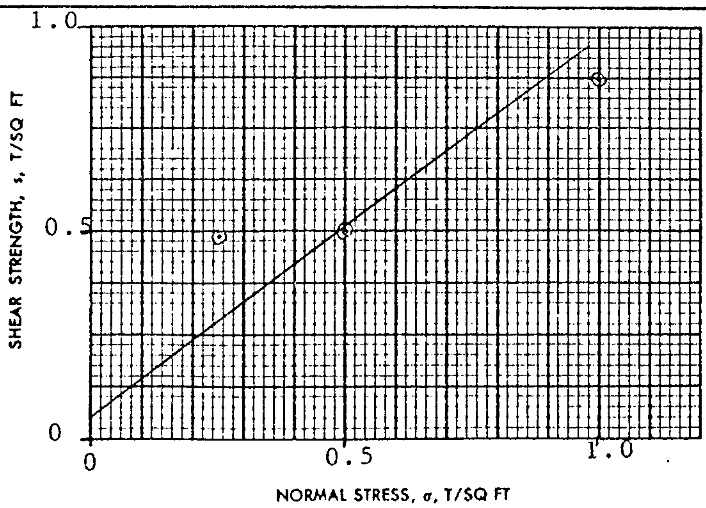
DATE January 1988

DIRECT SHEAR TEST REPORT

PLATE 22

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 OF—214-945 PLATE IX-3





SHEAR STRENGTH PARAMETERS

ϕ' = 37.0

TAN ϕ' = 0.753

c' = 0.05 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			A	B	C	
INITIAL	WATER CONTENT	w_o	20.9%	20.8%	20.9%	%
	VOID RATIO	e_o	.727	.722	.725	
	SATURATION	S_o	79 %	79 %	80 %	%
	DRY DENSITY, LB/CU FT	γ_d	99.7	100.0	99.9	
VOID RATIO AFTER CONSOLIDATION		e_c	.728	.709	.699	
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	–	–	–	
FINAL	WATER CONTENT	w_f	27.6%	27.0%	25.7%	%
	VOID RATIO	e_f	.762	.746	.710	
	SATURATION	S_f	100 %	100 %	100 %	%
NORMAL STRESS, T/SQ FT		σ	0.25	0.50	1.00	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.49	0.50	0.88	
ACTUAL TIME TO FAILURE, MIN		t_f	327	330	340	
RATE OF STRAIN, IN./MIN			.0015	.0015	.0015	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	–	–	–	

TYPE OF SPECIMEN Remolded — 3.25 IN. SQUARE — 0.5 IN. THICK

CLASSIFICATION Sandy Silt (ML)

LL 31 | PL 27 | PI 4 | G_s 2.76

REMARKS Div.No."Composite B" (4A, 4B&5B)

Dry density = 100pcf

PROJECT Sacramento River Flood Control System Evaluation (American River)

AREA

BORING NO. 4F-87 SAMPLE NO.

DEPTH EL DATE January 1988

DIRECT SHEAR TEST REPORT

PLATE 23

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO: 1966 OF—214-945 PLATE IX-3

APPENDIX M-2-F

REPORT OF SOIL TESTS - SACRAMENTO RIVER FLOOD
CONTROL SYSTEM EVALUATION (AMERICAN RIVER) - MAY 1988

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

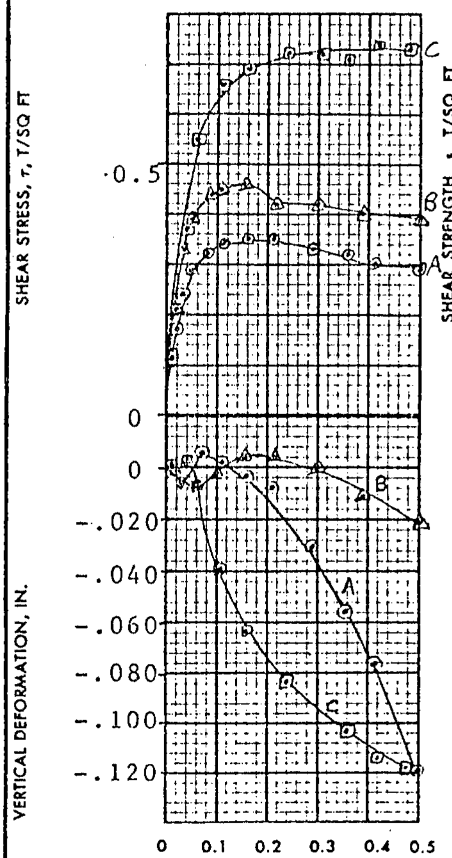
SOIL TEST RESULT SUMMARY

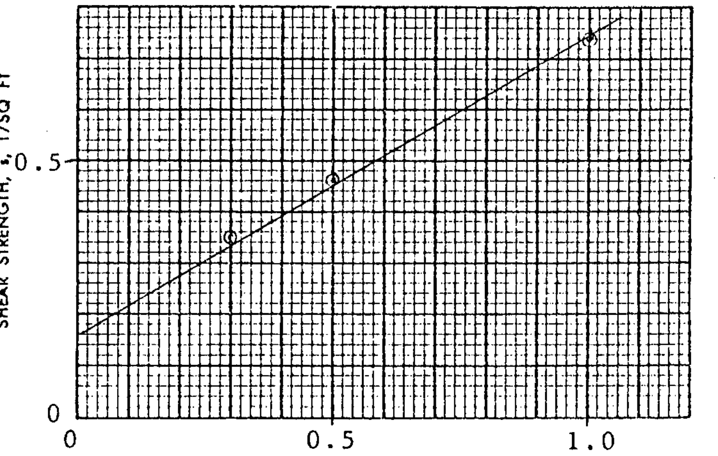
PROJECT: Sacramento River Flood Control System Evaluation

DATE: May 1988

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Gravel 24 12	Gravel 6 3	2 11/2	1 3/4	1/2 3/8	#4	#10	Sand #40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
105439	4F-88-1		3.0	3.7	Silt (ML)	GS = 2.75							100	99	98	90	35	9	
105440	4F-88-2		2.5	3.2	Lean clay with sand (CL)	GS = 2.73						100	97	95	92	85	35	12	--

SPD FORM 66A





SHEAR STRENGTH PARAMETERS

ϕ' = 29.5

TAN ϕ' = 0.566

c' = 0.16 T/SQ FT

☐ CONTROLLED STRESS

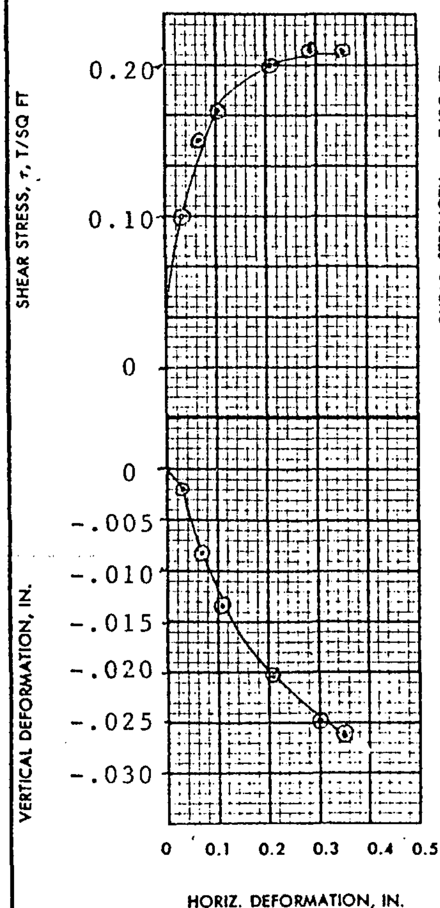
☒ CONTROLLED STRAIN

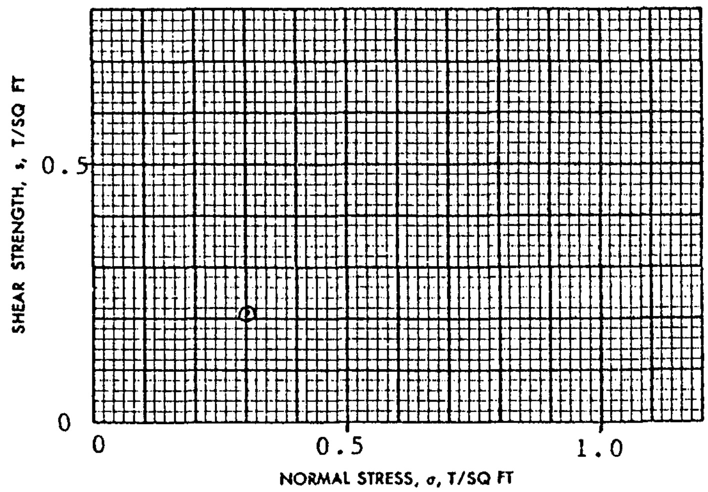
TEST NO.			A	B	C	
INITIAL	WATER CONTENT	w_o	21.8 %	24.9 %	26.8 %	%
	VOID RATIO	e_o	0.765	0.838	0.957	
	SATURATION	S_o	78 %	81 %	77 %	%
	DRY DENSITY, LB/CU FT	γ_d	96.5	92.7	87.0	
VOID RATIO AFTER CONSOLIDATION		e_c	0.752	0.822	0.925	
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	–	–	–	
FINAL	WATER CONTENT	w_f	26.0 %	29.8 %	32.2 %	%
	VOID RATIO	e_f	0.709	0.814	0.880	
	SATURATION	S_f	100 %	100 %	100 %	%
NORMAL STRESS, T/SQ FT		σ	0.3	0.5	1.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.35	0.46	0.74	
ACTUAL TIME TO FAILURE, MIN		t_f	150	200	480	
RATE OF STRAIN, IN./MIN			.0010	.0010	.0009	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	–	–	–	

TYPE OF SPECIMEN	UNDISTURBED	3.25 IN. SQUARE		0.5 IN. THICK	
CLASSIFICATION	LEAN CLAY with SAND (CL)				
LL 35	PL 23	PI 12		G_s 2.73	

REMARKS	PROJECT	SACRAMENTO RIVER FLOOD CONTROL
	AREA	–
	BORING NO. 4F88-2	SAMPLE NO. –
	DEPTH/EL 2.5'-3.2'	DATE MAY 1988
	DIRECT SHEAR TEST REPORT	

ENG FORM 1 JUN 65 **2092** *(EM 1110-2-1906)* PREVIOUS EDITIONS ARE OBSOLETE *(TRANSLUCENT)* GPO : 1966 OF—214-945





SHEAR STRENGTH PARAMETERS

ϕ' = ____________

TAN ϕ' = ____________

c' = ____________ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

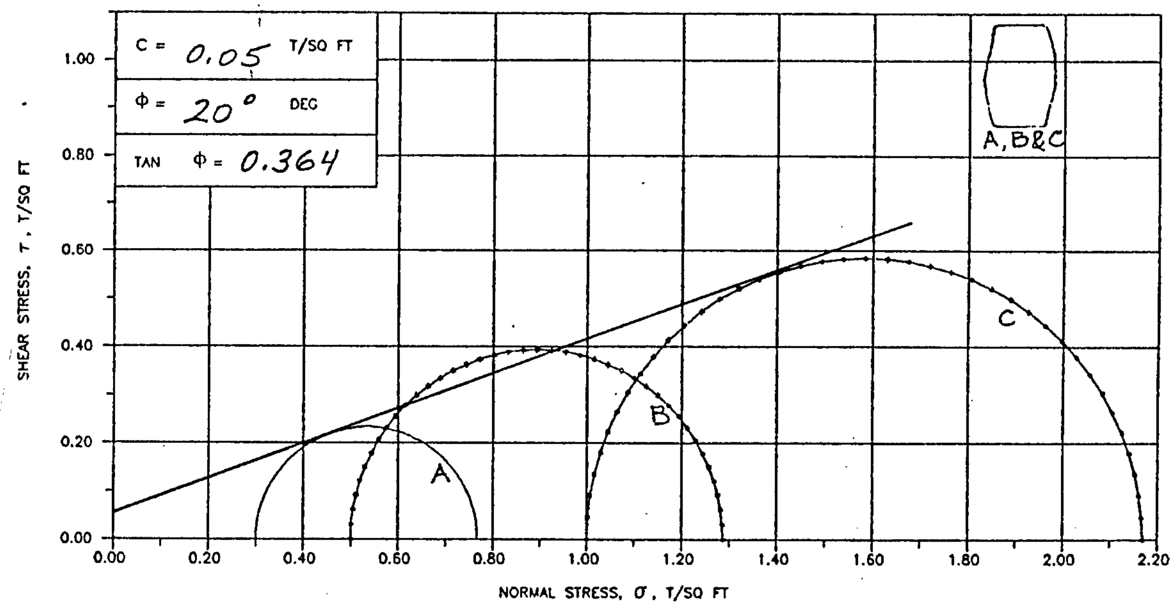
TEST NO.							
INITIAL	WATER CONTENT	w_o	16.6 %	%	%	%
INITIAL	VOID RATIO	e_o	0.959			
INITIAL	SATURATION	S_o	47 %	%	%	%
INITIAL	DRY DENSITY, LB/CU FT	γ_d	87.0			
VOID RATIO AFTER CONSOLIDATION		e_c	0.879			
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	-			
FINAL	WATER CONTENT	w_f	26.3 %	%	%	%
FINAL	VOID RATIO	e_f	0.759			
FINAL	SATURATION	S_f	95 %	%	%	%
NORMAL STRESS, T/SQ FT		σ	0.3			
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.21			
ACTUAL TIME TO FAILURE, MIN		t_f	472			
RATE OF STRAIN, IN./MIN			.0006			
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	-			

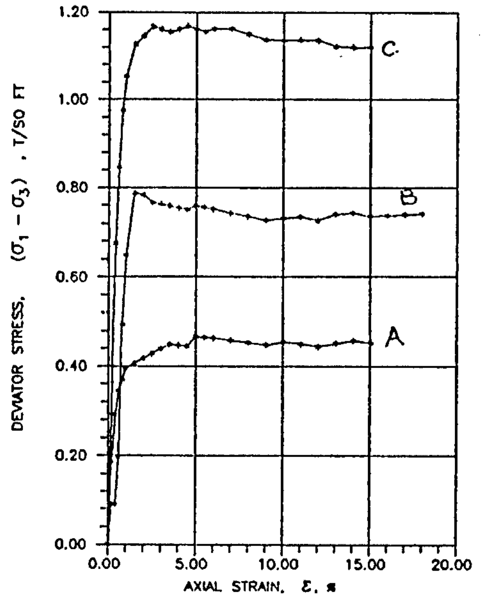
TYPE OF SPECIMEN UNDISTURBED		3.25 IN. SQUARE	0.5 IN. THICK
CLASSIFICATION LEAN CLAY with SAND (CL)			
LL 35	PL 23	PI 12		G_s 2.73

REMARKS

PROJECT	SACRAMENTO RIVER FLOOD CONTROL
AREA	-
BORING NO. 4F88-2	SAMPLE NO. -
DEPTH EL 2.5'-3.2'	DATE MAY 1988

DIRECT SHEAR TEST REPORT





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	19.5	19.3	19.2
	DRY DENSITY LB/CU FT	γ_{d_o}	80.8	86.5	86.5
	SATURATION, %	S_o	48	54	54
	VOID RATIO	e_o	1.12	0.984	0.985
BEFORE SHEAR	WATER CONTENT, %	w_c	37.8	34.2	33.0
	DRY DENSITY LB/CU FT	γ_{d_c}	84.2	88.4	90.0
	SATURATION, %	S_c	100	100	100
	VOID RATIO	e_c	1.04	0.942	0.907
	FINAL BACK PRESSURE, T/SQ FT	u_o	5.8	5.8	5.8
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.3	0.5	1.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.47	0.79	1.17
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	54	12	26
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	0.44	0.73	1.12
INITIAL DIAMETER, IN.		D_o	1.4	1.4	1.4
INITIAL HEIGHT, IN.		H_o	3.22	3.22	3.22

CONTROLLED STRAIN TEST

DESCRIPTION OF SPECIMEN Silt(ML)

LL 35	PL 26	PI 9	G_s 2.75	TYPE OF SPECIMEN Undistrubed	TYPE OF TEST R-Bar

REMARKS Div. No. 105439

PROJECT SACRAMENTO RIVER
FLOOD CONTROL SYSTEM EVALUATION

BORING NO. 4F-88-1 SAMPLE NO.

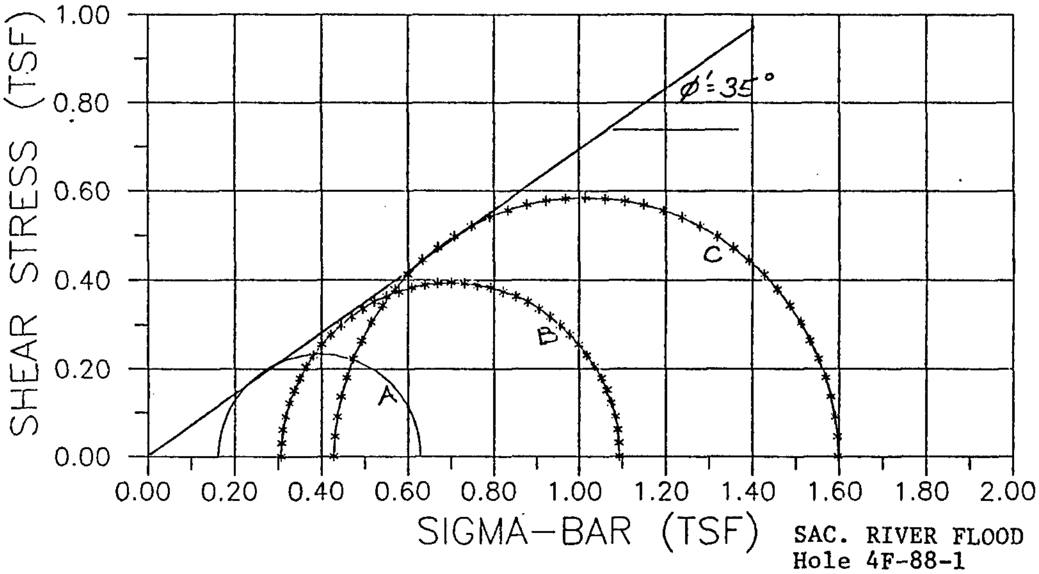
Depth: 3' - 3.7'

LABORATORY SPD DATE May 1988

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV. JUNE 1970

PLATE 4



SAC. RIVER FLOOD CONTROL
Hole 4F-88-1
Depth: 3' -3.7'
Div. No. 105439

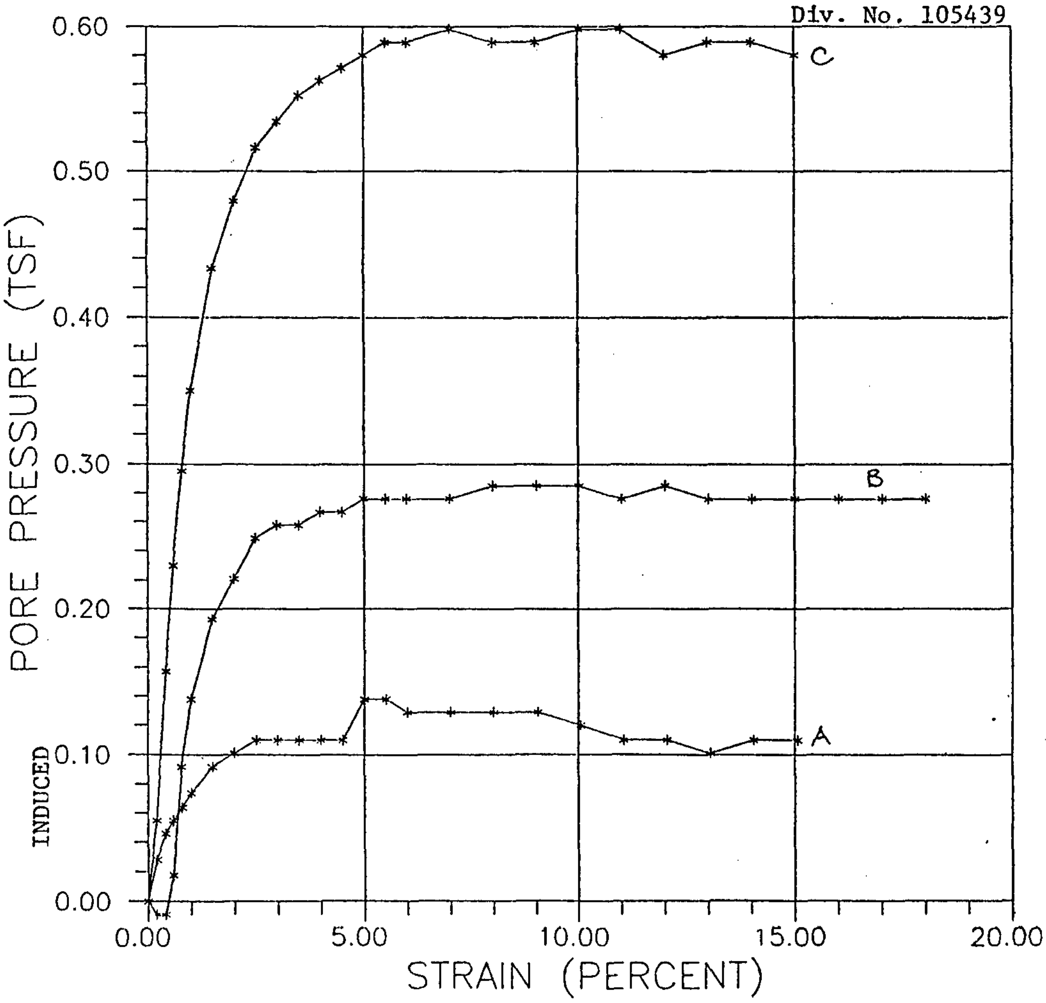


PLATE 5

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

Contract #2 ARFCD & MA 10 & 11

SOIL TEST RESULT SUMMARY

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION

DATE June 1987

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel					Sand					Fines	Liquid Limit	Plasticity Index	Field Moist. %
								3/4	1/2	3/8	#4	#10	#40	#60	#100	#200			
101530	DH-5	B-1	3.5	6.5	** Silty Sand (SM)			100	99	99	99	99	97	88	65	37 -	-	NP	12.7
101532	"	B-3	24.5	27.5	Silty Sand (SP-SM)							100	87	57	25	10			
101534	DH-6	B-2	9.5	12.5	Silty Sand (SM)							100	96	84	66	41			
101535	"	B-3	12.5	15.5	Silty Sand (SM)			100	99	99	98	98	93	80	63	43			15.7
101537	DH-7	B-1	1.5	7.5	** Silty Sand (SM)							100	99	93	74	49		NP	
101541	"	B-5	13.5	22.5	** Silty Sand (SM)							100	99	94	76	46		NP	
101543	DH-7A	B-1	1.5	15.0	Silty Sand (SM)							100	99	88	64	35		NP	
101545	DH-8	B-1	1.5	12.0	Silty Sand (SM)								100	96	78	50			
101546	"	B-2	12.0	20.4	Silty Sand (SM)							100	97	79	54	29		NP	

SPD Form 66A
1 May 83

PLATE 8, CONT. #2

U.S. ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

Contract #2 ARFCD & MA 10 & 11 — SOIL TEST RESULT SUMMARY

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION — DATE June 1987

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation		Laboratory Descriptive Classification	Mechanical Analysis-% Finer												Liquid Limit	Plasticity Index	Field Moist. %
			From	To		Gravel					Sand					Fines				
											#4	#10	#40	#60	#100	#200				
101548	DH-9	B-1	1.5	18.0	*Sandy Silt(ML)							100	99	96	86	62	26	2		
101549	"	B-2	18.5	25.5	Silty Sand(SM)							100	99	88	70	46		NP		
101550	DH-10	B-1	1.5	10.5	*Sandy Clayey Silt(ML)								100	99	95	80	31	6		
101552	"	B-3	12	18.0	Silty Sand (SM)								100	99	88	39				
101553	"	B-4	19.5	31.5	*Sandy Silt(ML)							100	99	98	91	70	23	1		
101555	DH-11	B-1	1.5	14.5	**Sandy Silt(ML)								100	99	94	79		NP		
101558	"	B-4	25.5	36.0	**Sandy Silt(ML)							100	99	94	83	79		NP		
101561	DH-12	B-2	6.0	10.5	Sandy Clay(CL)						100	99	90	80	71	58	26	11		
101564	DH-12	B-5	18.5	22.0	Sandy Clay(CL)							100	92	83	74	57	25	12		
101565	"	B-6	21.5	25.5	Clayey Sand (SC-SM)						100	99	91	84	72	50	26	6		
101566	"	B-7	26.0	34.5	Silty Sand (SM)							100	82	60	44	28				

SPD Form 66A
1 May 83

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

Contract #2 ARFCD * MA 10 & 11 SOIL TEST RESULT SUMMARY

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION DATE June 1987

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel					Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
101567	DH-12A	B-1	1.5	20.0	Sandy Silt(ML)							100	99	99	98	85	28	2	
101570	DH-13	S-1	7.5	9.5													45	21	
101572	"	B-4	12.5	18.5													51	31	
101574	"	S-2	21.5	23													38	25	
101580	DH-14	B-2	9.0	12	Sandy Silty Clay(CL)						100	99	93	89	85	71	38	15	
101583	"	B-5	21	24	Sandy Silt(ML)						100	99	95	87	75	54	26	4	
101584	"	B-6	24	29													38	14	
101585	"	B-7	31	34.5	Sandy Silt(ML)							100	98	97	89	71		NP	
101587	"	B-9	40.5	45								100	99	98	72	18			
101590	DH-14A	B-3	7.0	11	Sandy Clayey Silt(ML)						100	98	94	90	84	64	32	9	
101591	DH-14A	B-4	11.5	15								100	98	96	85	63			

NOT IN THIS STUDY

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1 May 83

PLATE 10, CONT. #2

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

Contract #2 ARFCD & MA 10 & 11

SOIL TEST RESULT SUMMARY

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION

DATE June 1987

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation		Laboratory Descriptive Classification	Mechanical Analysis-% Finer												Liquid Limit	Plasticity Index	Field Moist. %
						Gravel					Sand						Fines			
			From	To							#4	#10	#40	#60	#100		#200			
101616	DH-18	B-2	7.5	10.5	Sandy Clay(CL)						100	95	80	71	63		54	36	14	
101617	"	B-3	10.5	15														38	14	
101619	"	B-5	19.5	22.5	Sandy Silt(ML)						100	94	82	77	71		61	36	9	
101622	DH-19	S-1	6	8.5														25	12	
101624	"	B-3	13	16	Clayey Sand (SC-SM)						100	96	81	73	63		49	27	6	
101628	DH-19A	S-1	7.5	9	Sandy Clayey Silt(ML)						100	99	90	84	77		67	29	5	
101633	DH-20	B-2	6	9														27	13	
101634	"	S-1	10.5	12.5	Clay (CL)													51	36	
101636	"	B-4	17.5	19	Silty Sand(SM)						100	99	62	33	21		16			
101638	DH-21	B-2	14	17	Silty Sand (SM)							100	98	92	75		33			
101639	"	B-3	24	25.5	Silty Sand(SM)							100	99	94	76		45		NP	

NOT IN THIS STUDY

SPD Form 66A
1 May 83

PLATE 12, CONT. #2

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

Contract #2 ARFCD & MA 10 & 11 SOIL TEST RESULT SUMMARY

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION DATE June 1987

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer	Gravel 1	Gravel 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
101641	DH-22	B-1	3.5	6.5	Sand (SP)							100	66	7	6	3 -	--		
101643	"	B-3	20	23	Sandy Silt(ML)								100	99	98	88	34	4	
101645	DH-23	B-1	6	9	Sand(SP)						100	99	43	10	6	4			3.7
101648	"	B-4	25.5	28.5	Silty Sand (SM)							100	99	93	70	29			
101649	"	B-5	30	33	Silty Sand (SP-SM)							100	98	70	28	11			
101650	DH-23A	B-1	22	25	Sand (SP)							100	92	33	9	5			
101651	"	B-2	29.5	32.5	Silty Sand (SP-SM)							100	59	25	12	7			
101652	DH-24	B-1	3.5	6.5	Sand(SP)		100	98	98	98	97	96	47	15	7	4			3.1
101654	"	B-3	23	26	*Sandy Silt(ML)								100	99	96	76		NP	

SPD Form 66A
1 May 83

PLATE 13, CONT. #2

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

Contract #2 ARFCD & MA 10 & 11

SOIL TEST RESULT SUMMARY

PROJECT SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION

DATE June 1987

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer	Gravel 1	Gravel 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
101657	DH-25	B-2	6.9	9.5	Sandy Silt (ML)							100	92	84	73	51	--		
101658	"	B-3	12.5	15.5	Sand (SP)							100	66	26	8	3			
101660	"	S-1	29	31.3	Sandy Silt (ML)							100	99	99	89	58		NP	
101662	DH-26	B-2	11	14	Silty Sand (SM)							100	71	33	20	13			
101663	"	B-3	18.5	21.5	Sand (SP)						100	99	52	10	5	3			3.1
101664	"	B-4	33.5	36.5	Sand (SP)							100	61	20	9	5			
101665	DH-27	B-1	2	5	Silty Sand (SM)							100	94	72	44	24			
101667	"	S-1	29	31.3	**Sandy Silt (ML)								100	99	92	64		NP	
101669	DH-28	B-1	3.5	6.5	Sandy Silt (ML)		100	99	98	97	96	96	93	86	75	56	26	4	14.8
101670	"	B-2	9.5	12.5	Silty Sand (SM)				100	99	99	97	60	24	17	13			4.0
101673	"	S-1	27.4	29.7	*Sandy Silt (ML)								100	98	91	78	24	1	

SPD Form 66A
1 May 83

PLATE 14, CONT. #2

TABLE I

STEADY SEEPAGE EXIT HEIGHT ABOVE LANDSIDE TOE - (y)

X-section	LEFT BANK						RIGHT BANK					
	115,000 cfs		130,000 cfs		180,000 cfs		115,000 cfs		130,000 cfs		180,000 cfs	
	x,m,h	y	x,m,h	y	x,m,h	y	x,m,h	y	x,m,h	y	x,m,h	y
1	0	0	0	0	0	0	177,42,13.2	1.0	175,44,13.3	1.0	173,45,13.6	1.1
2	0	0	0	0	0	0	120,51,13	1.1	118,52,13.2	1.5	117,54,13.6	1.6
3	57,4,2.6	.13	56,5,2.9	.16	55,7,3.7	.27	119,34,13.3	1.5	117,35,13.6	1.6	115,38,14.4	1.8
4	68,6,1.9	.06	66,8,2.4	.09	64,10,3.3	.18	118,31,13.5	1.6	115,34,14	1.8	113,41,14.9	2.0
5	79,12,4.4	.26	77,14,5	.35	74,17,6.1	.53	109,46,13.7	1.7	107,48,14.3	1.9	105,50,15.4	2.2
6	79,6,2.2	.07	76,9,2.9	.12	70,15,4.2	.27	129,56,14	1.5	126,58,14.7	1.7	122,63,16	2.0
7	69,5,2.1	.07	67,8,2.9	.14	62,12,4.4	.33	87,34,12.3	1.8	86,36,13.1	2.0	81,41,14.6	2.6
8	93,11,5.4	.34	91,12,6.2	.46	87,16,7.9	.76	130,35,12.4	1.2	127,37,13.2	1.4	123,41,14.9	1.8
9	92,7,3.4	.14	90,9,4.3	.22	84,15,6.2	.49	65,9,3.7	.23	63 11,4.6	.36	58,15,6.5	.76
10	0	0	0	0	0	0	68,22,7.9	.94	66,25,8.9	1.2	60,30,11.2	2.1
11	78,20,5.8	.45	74,24,6.9	.66	67,31,9.4		75,15,7.2	.73	73,17,8.3	1.0	68,23,10.8	1.8
12	0	0	0	0	0	0	67,16,6.4	.64	65,17,7.5	.90	62,20,10.1	1.7
13	0	0	0	0	0	0	73,18,6.9	.68	70,21,8	.95	64,27,10.7	.95
14	0	0	0	0	0	0	81,24,7.4	.70	76,28,8.6	.99	68,36,11.3	1.8
15	0	0	0	0	0	0	78,20,8.2	.90	75,22,9.3	1.2	69,29,12.1	2.1
16	92,15,3.8	.17	87,20,4.9	.29	78,30,7.8	.79	93,21,10.9	1.3	90,24,12	1.7	86,29,14.9	2.7
17	93,13,2.8	.09	88,19,3.9	.18	77,29,6.9	.62	102,21,9.1	.86	99,25,10.2	1.1	91,33,13.2	2.0
18	83,11,4.2	.23	79,15,5.5	.41	71,23,8.6	1.1	73,16,5.2	.39	68,21,6.5	.64	60,29,9.6	1.5
19	84,15,4.8	.29	80,20,6.0	.47	72,27,9.2	1.2	99,20,5.4	.31	94,24,6.6	.48	85,33,9.8	1.2
20	84,10,2.8	.10	81,13,4.1	.22	71,23,7.3	.77	73,16,4.1	.24	69,20,5.4	.44	60,30,8.6	1.2
21	80,11,3.8	.19	76,15,5.1	.36	70,21,8.3	1.0	77,16,5.6	.43	73,20,6.9	.68	67,27,10.1	1.5
22	72,1.0,.6	.01	69,4,1.9	.06	58,14,5.2	.49	58,3,.6	.01	55,8,1.9	.07	41,23,5.2	.64
23	0	0	0	0	0	0	72,8,2.6	.10	65,15,3.8	.23	51,29,7.2	.98
24	96,3,1.4	.02	91,8,2.8	.09	79,20,6.1	.49	55,7,2.2	.09	51,11,3.5	.25	39,22,6.9	1.2
25	115,5,1.5	.02	112,8,2.8	.08	103,17,6.1	.39	56,5,1.2	.03	51,10,2.5	.13	37,24,5.8	.86
26	79,3,1.1	.02	75,8,2.4	.08	66,17,5.8	.53	57,12,3.7	.25	52,17,5	.49	39,30,6.8	1.1
27	116,10,2.8	.07	94,23,4.1	.19	71,45,7.7	.79	78,25,4.8	.49	71,32,6.1	.52	49,53,9.7	1.6
28	77,5,1.6	.04	72,10,3	.13	61,22,6.9	.79	60,11,3.3	.19	56,15,4.7	.41	43,29,8.6	1.6
29	79,3,.6	.01	73,8,2	.06	60,21,6.1	.63	70,6,1.3	.03	64,12,2.7	.12	46,30,6.8	.95
30	71,8,3.0	.14	67,11,4.6	.34	56,22,8.9	1.4	57,11,2.2	.09	52,16,3.8	.29	37,31,8.1	1.6
31	61,4,1.0	.02	58,7,2.6	.13	46,19,7.1	1.1	61,10,2.3	.09	57,14,3.9	.28	39,32,8.4	1.7
32	0	0	0	0	70,15,4.5	.30	97,16,4.0	.18	87,25,5.6	.37	72,41,10.2	1.4
33	68,1.0,.1	0	62,5,.7	.01	48,20,6.5	.88	65,11,2.3	.09	58,19,3.9	.27	42,34,8.7	1.7
34	58,2.0,1.1	.02	51,10,1.8	.07	38,23,7.6	1.5	68,7,2.4	.09	61,15,4.1	.29	46,29,8.9	1.6
35	0	0	59,2,.2	0	44,16,5.1	.60	63,5,1.6	.04	58,10,3.3	.20	44,25,8.2	1.5
36	69,2.0,.8	.01	66,5,2.5	.10	57,14,7.5	1.0	69,5,1.2	.02	62,10,2.9	.14	46,27,7.9	1.3
37	0	0	0	0	48,13,5	.54	72,15,3.1	.14	65,22,4.8	.36	46,41,9.8	1.9
38	0	0	47,5,.7	.01	38,9,5.8	.94	73,18,4.8	.33	68,24,6.5	.63	50,42,11.6	2.5
39	64,2,.1	0	59,5,1.8	.06	42,22,6.9	1.1	0	0	82,5,1.1	.02	65,21,6.2	.61
40	0	0	61,4,1.5	.04	46,19,6.6	.95	0	0	72,6,1.1	.02	54,24,6.2	.71
41	0	0	80,1,.2	0	56,32,5.4	.50	65,4,1.1	.02	59,10,2.8	.14	40,29,8	1.5
42	0	0	0	0	46,15,4.0	.36	0	0	57,7,1.6	.05	43,22,6.7	1.0
43	0	0	0	0	43,10,3.6	.32	67,9,2.5	.10	60,16,4.4	.34	44,31,9.6	2.0
44	0	0	0	0	49,15,4.4	.41	63,3,.7	.01	57,9,2.5	.12	38,27,7.8	1.5

Note: all dimensions given are in feet

$d = 0.3m + x$

$S_0 = \sqrt{d^2 + h^2}$

$a = S_0 - \sqrt{S_0^2 - \frac{h^2}{\sin^2\beta}}$

$y = a\sin\beta$

where y is the seepage exit height

L. Casagrande method (Ref. 5)

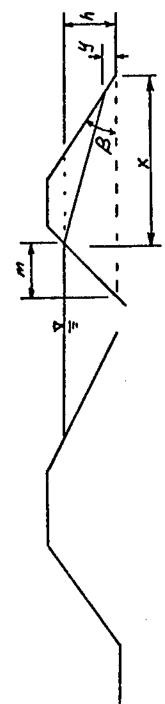


TABLE 2 - LEVEE HEIGHT, FREEBOARD AND STABILITY SUMMARY(1)

DWR X-SECTION	APPROXIMATE RIVER MILE	LEVEE HEIGHT (FT) LEFT	LEVEE HEIGHT (FT) RIGHT	FREEBOARD(2) LEFT BANK	FREEBOARD(2) RIGHT BANK	HEAD/SEEPAGE EXIT HEIGHTS(3) Q=115,000 cfs LEFT BANK	Q=115,000 cfs RIGHT(4) BANK	Q=130,000 cfs LEFT BANK	Q=130,000 cfs RIGHT(4) BANK	Q=180,000 cfs LEFT BANK	Q=180,000 cfs RIGHT(4) BANK
1	0.0	0	20.9	8.7/8.6/8.3	7.7/7.6/7.3	0/0	13.2/1.0	0/0	13.3/1.0	0/0	13.6/1.1
2	0.2	0	21*	10.8/10.6/10.2	8.3/8.1/7.7	0/0	13.0*/1.1	0/0	13.2*/1.5	0/0	13.6*/1.6
3	0.5	9.9	20*	7.3/7.0/6.2	6.3/6.0/5.2	2.6/.13	13.3*/1.5	2.9/.16	13.6*/1.6	3.7/.27	14.4*/1.8
4	0.8	9.6	20*	7.7/7.2/6.3	6.2/5.7/5.0	1.9/.06	13.5*/1.6	2.4/.09	14.0*/1.8	3.3/.18	14.9*/2.0
5	1.0	11.3	20*	6.9/6.3/5.2	6.5/5.9/4.8	4.4/.26	13.7*/1.7	5.0/.35	14.3*/1.9	6.1/.53	15.4*/2.2
6	1.3	9.3	20*	7.1/6.4/5.1	6.5/5.8/4.5	2.2/.07	14.0*/1.5	2.9/.12	14.7*/1.7	4.2/.27	16.0*/2.0
7	1.5	9.4	19.0	7.3/6.5/5.0	6.7/5.9/4.4	2.1/.07	12.3/1.8	2.9/.14	13.1/2.0	4.4/.33	14.6/2.6
8	1.8	11.7	20*	6.3/5.5/3.8	7.7/6.9/5.2	5.4/.34	12.4*/1.2	6.2/.46	13.2*/1.4	7.9/.76	14.9*/1.8
9	2.0	13.9	12.2	10.5/9.6/7.7	8.5/7.6/5.7	3.4/.14	3.7/.23	4.3/.22	4.6/.36	6.2/.49	6.5/.76
10	2.2	4.3	16.7	9.8/8.8/6.5	8.8/7.8/5.5	0/0	7.9/.94	0/0	8.9/1.2	0/0	11.2/2.1
11	2.5	14.9	15.8	9.1/8.0/5.5	8.6/7.5/5.0	5.8/.45	7.2/.73	6.9/.66	8.3/1.0	9.4/1.3	10.8/1.8
12	2.8	2.4	15.1	8.5/7.4/5.8	8.7/7.6/6.0	0/0	6.4/.64	0/0	7.5/.9	0/0	10.1/1.7
13	3.0	0	15.5	10.3/9.2/6.5	8.6/7.5/4.8	0/0	6.9/.68	0/0	8.0/.95	0/0	10.7/.95
14	3.2	0	16.3	10.4/9.2/6.5	8.9/7.7/5.0	0/0	7.4/.70	0/0	8.6/.99	0/0	11.3/1.8
15	3.6	0	17.7	16.2/15.1/12.3	9.5/8.4/5.6	0/0	8.2/.90	0/0	9.3/1.2	0/0	12.1/2.1
16	3.8	15.2	20.9	11.4/10.3/7.4	10.0/8.9/6.0	3.8/.17	10.9/1.3	4.9/.29	12.0/1.7	7.8/.79	14.9/2.7
17	4.0	14.4	18.8	11.6/10.5/7.0	9.7/8.6/5.6	2.8/.09	9.1/.86	3.9/.18	10.2/1.1	6.9/.62	13.2/2.0
18	4.2	16.0	14.9	11.8/10.5/7.4	9.7/8.4/5.5	4.2/.23	5.2/.39	5.5/.41	6.5/.64	8.6/1.1	9.6/1.5
19	4.5	15.6	13.2	10.8/9.6/6.4	9.4/8.2/4.8	4.8/.29	5.4/.31	6.0/.47	6.6/.48	9.2/1.2	9.8/1.2
20	4.7	13.5	14.6	10.7/9.4/7.2	9.1/7.8/4.6	2.8/.10	4.1/.24	4.1/.22	5.4/.44	7.3/.77	8.6/1.2
21	4.9	14.7	14.6	10.9/9.6/7.4	9.0/7.7/4.5	3.8/.19	5.6/.43	5.1/.36	6.9/.68	8.3/1.0	10.1/1.5
22	5.2	12.3	7.4	11.7/10.4/7.1	6.8/5.5/2.2	0.6/.01	.6/.01	1.9/.06	1.9/.07	5.2/.47	5.2/.64
23	5.5	4.9	9.8	11.9/10.7/7.3	7.2/6.0/2.6	0/0	2.6/.10	0/0	3.8/.23	0/0	7.2/.98
24	5.8	12.6	8.8	11.2/9.9/6.0	6.6/5.3/1.4	1.4/.02	2.2/.09	2.7/.09	3.5/.25	6.1/.49	6.9/1.2
25	6.0	13.8	7.8	12.3/11.0/7.7	6.6/5.3/1.5	1.5/.02	1.2/.03	2.8/.08	2.5/.13	6.1/.39	5.8/.86
26	6.3	11.8	9.9	10.7/9.4/6.0	6.2/4.9/1.5	1.1/.02	3.7/.25	2.4/.08	5.0/.49	5.8/.53	8.4/1.1
27	6.5	12.8	11.0	10.0/8.7/5.1	6.2/4.9/1.3	2.8/.07	4.8/.49	4.1/.19	6.1/.52	7.7/.79	9.7/1.6
28	6.8	12.4	10.6	10.8/9.4/5.5	7.3/5.9/2.0	1.6/.04	3.3/.19	3.0/.13	4.7/.41	6.9/.79	8.6/1.6
29	7.0	11.0	8.0	10.4/9.0/4.9	6.7/5.3/1.2	0.6/.01	1.3/.03	2.0/.06	2.7/.12	6.1/.63	6.8/.95
30	7.2	12.0	9.2	9.0/7.4/3.1	7.6/5.4/1.7	3.0/.14	2.2/.09	4.6/.34	3.8/.29	8.9/1.4	8.1/1.6
31	7.4	9.2	9.1	8.2/6.6/2.1	6.8/5.2/0.7	1.0/.02	2.3/.09	2.6/.13	3.9/.28	7.1/1.1	8.4/1.7
32	7.8	11.4	11.5	13.1/11.5/6.9	7.5/5.9/1.3	0/0	4.0/.18	0/0	5.6/.37	4.5/.30	10.2/1.4
33	8.0	7.9	9.8	7.8/6.2/1.4	7.5/5.9/1.1	0.1/0	2.3/.09	.07/.01	3.9/.27	6.5/.88	8.7/1.7
34	8.3	9.2	10.6	8.1/6.4/1.6	8.2/6.5/1.7	1.1/.02	2.4/.09	1.8/.07	4.1/.29	7.6/1.5	8.9/1.6
35	8.5	6.8	10.1	8.3/6.6/1.7	8.5/6.8/1.9	0/0	1.6/.04	0.2/0	3.3/.20	5.1/.60	8.2/1.5
36	8.8	9.7	10.2	8.9/7.2/2.2	9.0/7.3/2.3	0.8/.01	1.2/.02	2.5/.10	2.9/.14	7.5/1.0	7.9/1.3
37	9.0	7.0	11.8	8.7/7.0/2.0	8.7/7.0/1.9	0/0	3.1/.14	0/0	4.8/.36	5.0/.54	9.8/1.9
38	9.2	7.7	13.5	8.7/7.0/1.9	8.6/7.0/1.8	0/0	4.8/.33	0.7/.01	6.5/.63	5.8/.94	11.6/2.5
39	9.5	9.1	8.3	9.0/7.3/2.2	8.9/7.2/2.1	0.1/0	0/0	1.8/.06	1.1/.02	6.9/1.1	6.2/.61
40	9.7	9.1	8.3	9.3/7.6/2.5	8.9/7.2/2.1	0/0	0/0	1.5/.04	1.1/.02	6.6/.95	6.2/.71
41	9.9	8.5	9.7	10.0/8.3/3.1	8.6/6.9/1.7	0/0	1.1/.02	0.2/0	2.8/.14	5.4/.50	8.0/1.5
42	10.2	7.7	9.0	10.6/8.8/3.7	9.2/7.4/2.3	0/0	0/0	0/0	1.6/.05	4.0/.36	6.7/1.0
43	10.5	8.7	11.2	10.7/8.8/3.6	8.7/6.8/1.6	0/0	2.5/.10	0/0	4.4/.34	3.6/.32	9.6/2.0
44	10.7	8.0	9.8	10.7/8.9/3.6	9.1/7.3/2.0	0/0	.7/.01	0/0	2.5/.12	4.4/.41	7.8/1.5
DH-31	10.9	7.0**	-								
DH-4	11.0	-	10.0**								
DH-3	11.8	-	6.0**								
DH-2	12.3	-	4.0**								
DH-1	12.5	-	4.0**								

* Estimated from California DWR cross sections and USGS topographic maps.
** Estimated from levee cross sections taken during borings.

NOTES:
(1) Numbers underlined in the table identify unstable parameters. Minimum freeboard - 3 feet; Maximum head (piping) - 6 feet; Maximum seepage exit elevation (stability) - 0.60 feet.
(2) Freeboard in feet for flows of 115,000/130,000/180,000 cfs.
(3) HEAD/SEEPAGE EXIT ELEVATION
HEAD: Elevation difference between river stage and the adjacent land surface. SEEPAGE EXIT HEIGHT: Estimated seepage exit height above landside levee toe.
(4) Right bank levee and foundation soils downstream of R.M. 5 are are fine grained and have a low permeability. Therefore, steady seepage conditions are unlikely to develop in this reach during

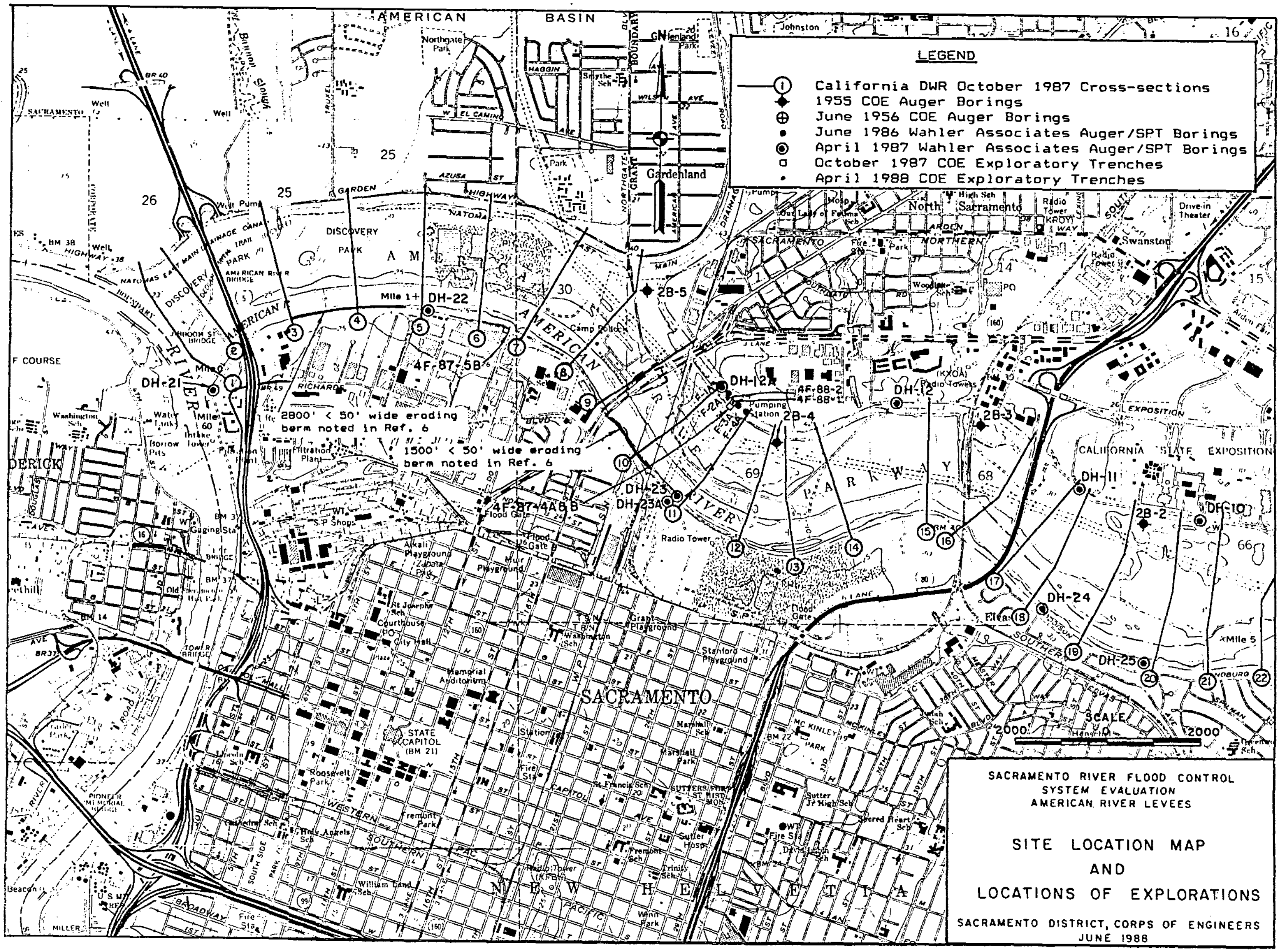

LEGEND
California DWR October 1987 Cross-sections
1955 COE Auger Borings
June 1956 COE Auger Borings
June 1986 Wahler Associates Auger/SPT Borings
April 1987 Wahler Associates Auger/SPT Borings
October 1987 COE Exploratory Trenches
April 1988 COE Exploratory Trenches
2800' < 50' wide eroding berm noted in Ref. 6
1500' < 50' wide eroding berm noted in Ref. 6
AMERICAN BASIN
AMERICAN RIVER
DISCOVERY PARK
SACRAMENTO
NORTH SACRAMENTO
CALIFORNIA STATE EXPOSITION
PARKWAY
NEW HELVETIA
DH-21
DH-22
DH-12A
DH-12
DH-11
DH-10
DH-23
DH-23A
DH-24
DH-25
2B-2
2B-3
2B-4
2B-5
4F-87-5B
4F-87-4A&B
4F-88-2
4F-88-1
SCALE
2000
0
2000
SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES
SITE LOCATION MAP
AND
LOCATIONS OF EXPLORATIONS
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

FIGURE 1

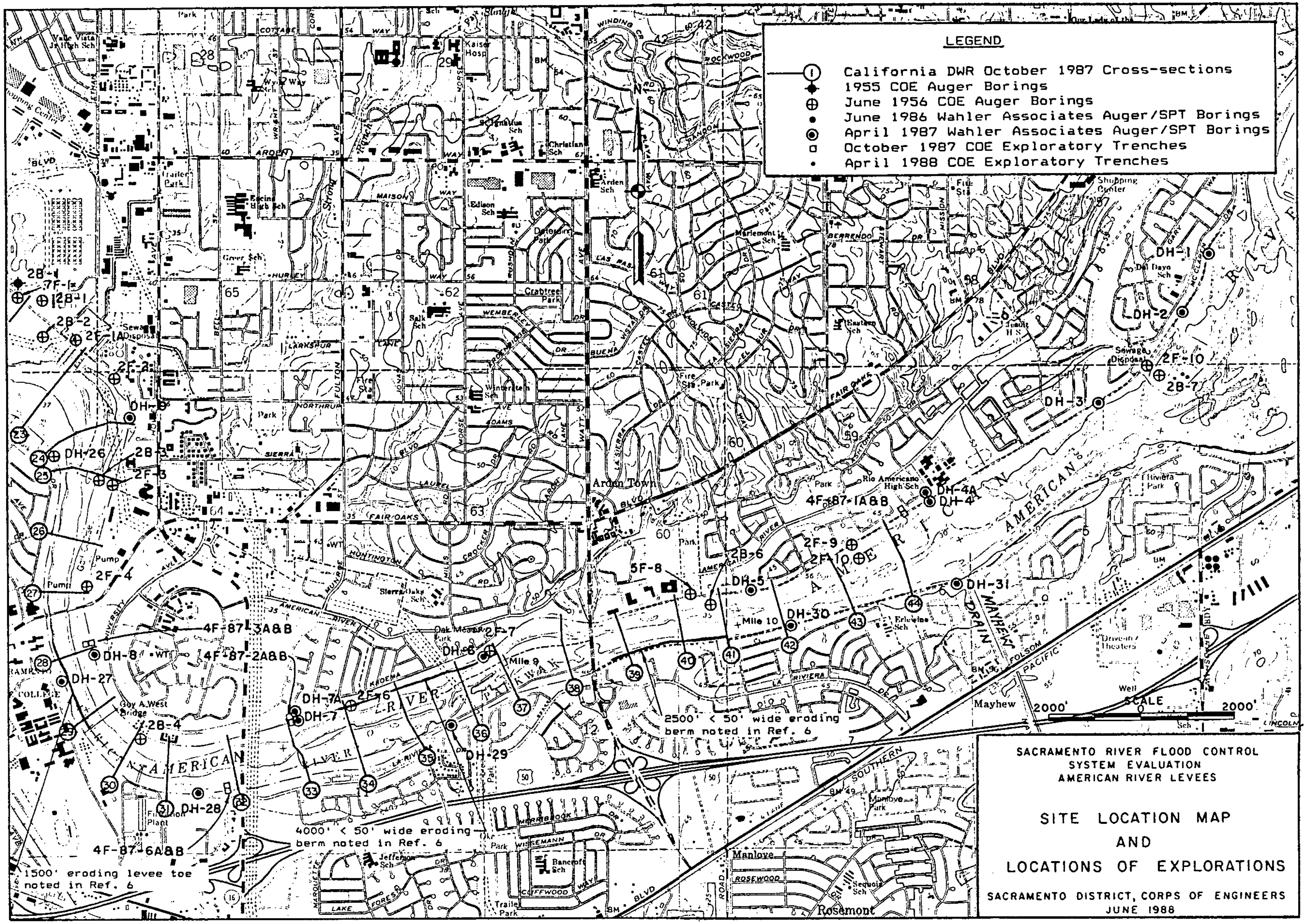
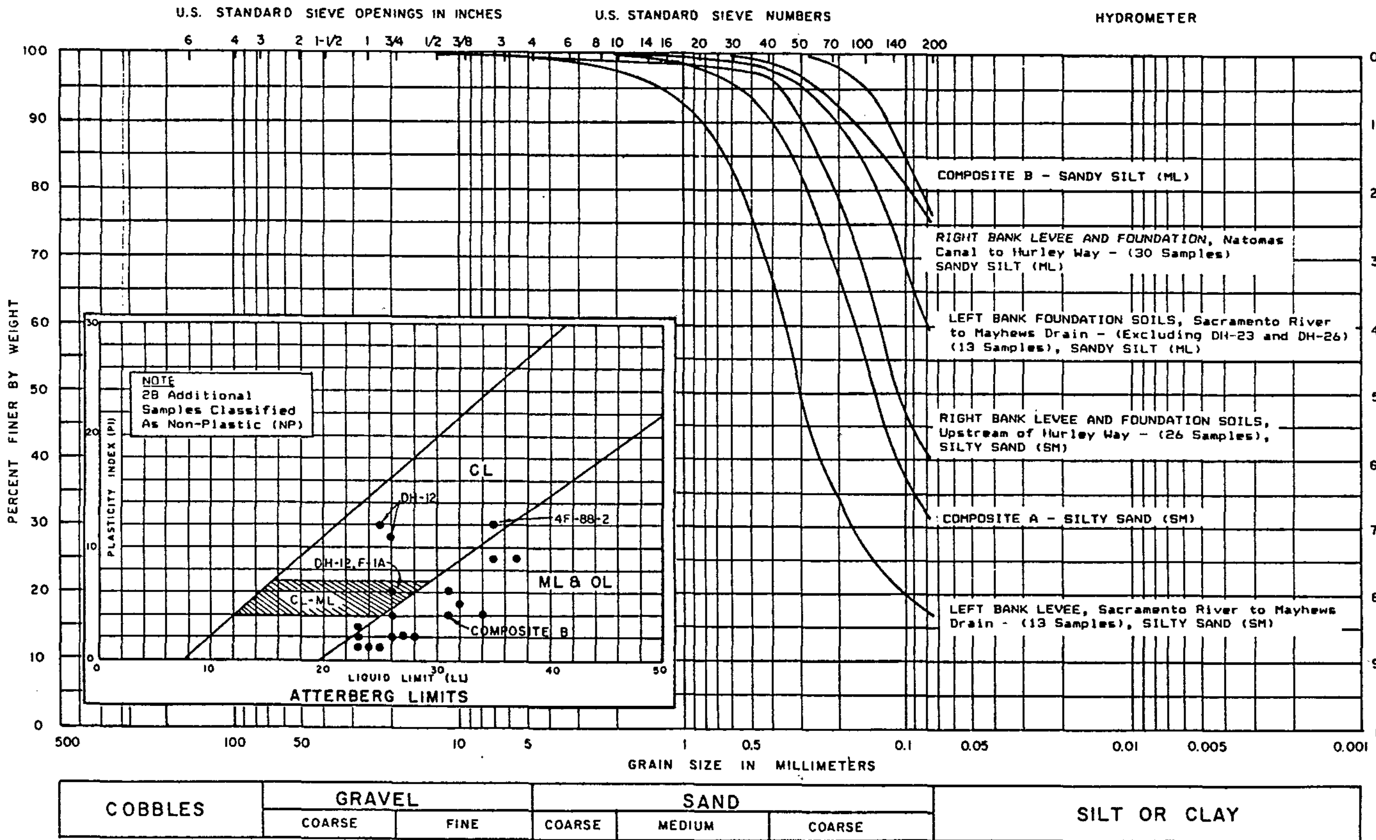


FIGURE 1 cont.



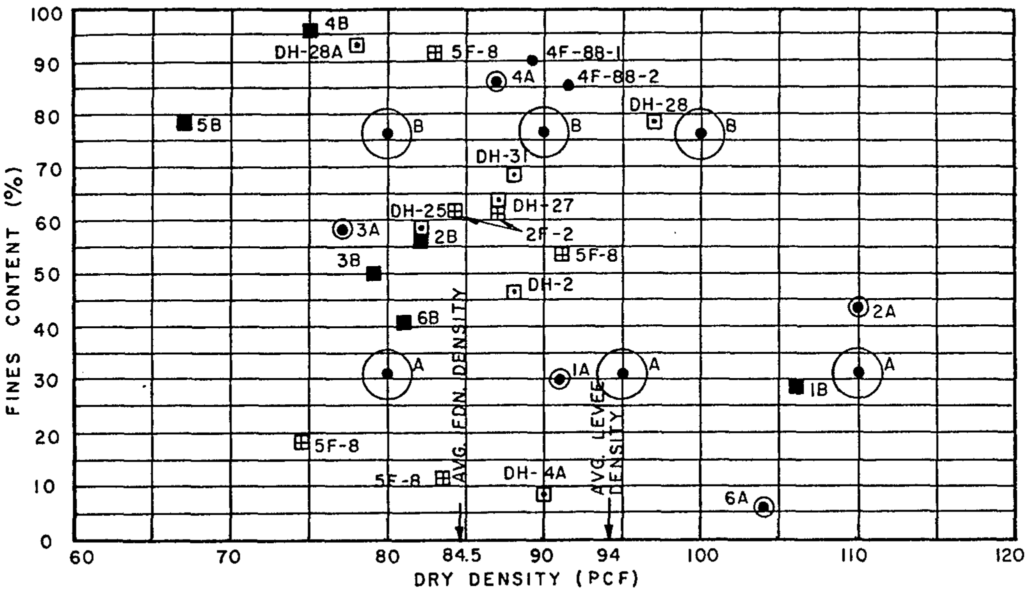
GRADATION CURVES

SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

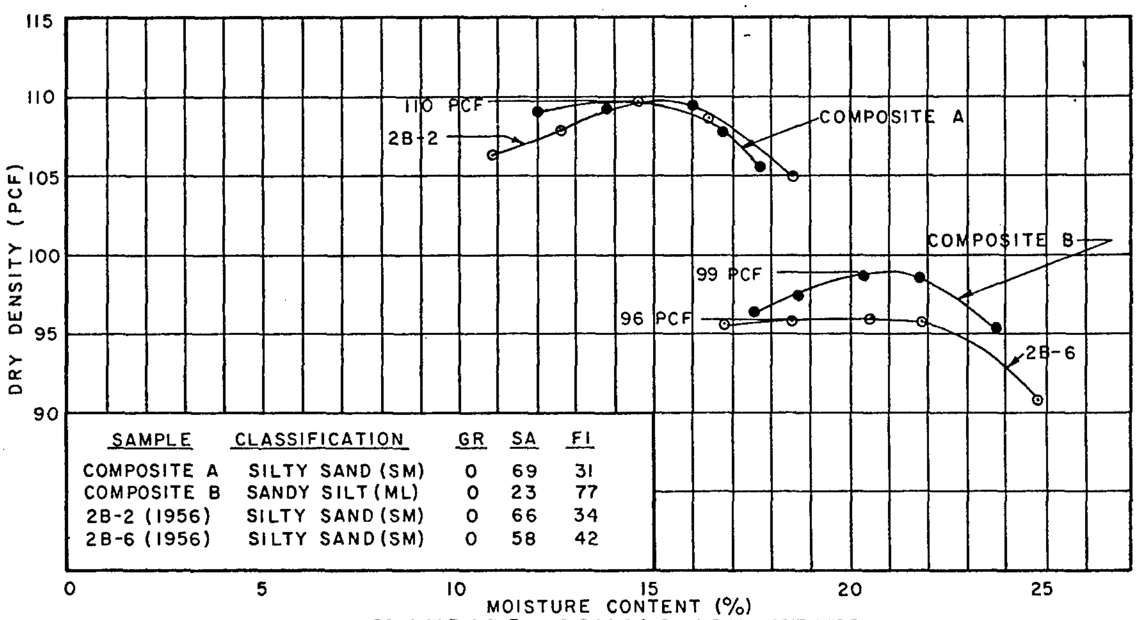
LABORATORY GRADATIONS
AND
ATTERBERG LIMITS

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

FIGURE 2



IN-SITU DENSITY TESTS(1)



SAMPLE	CLASSIFICATION	GR	SA	FI
COMPOSITE A	SILTY SAND (SM)	0	69	31
COMPOSITE B	SANDY SILT (ML)	0	23	77
2B-2 (1956)	SILTY SAND (SM)	0	66	34
2B-6 (1956)	SILTY SAND (SM)	0	58	42

STANDARD COMPACTION TESTS

(1) LEGEND

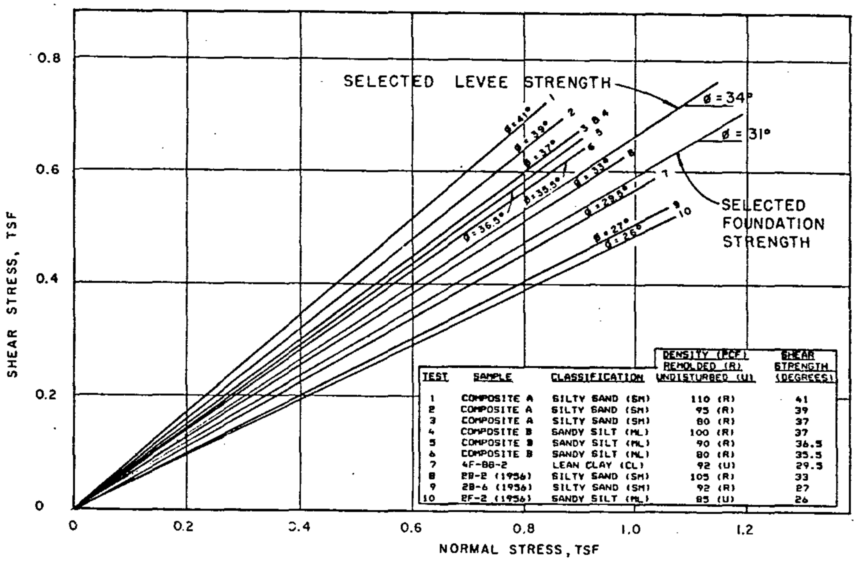
- ● LEVEE CHUNK DENSITY - OCT. 1956
- ◉ LEVEE SAND CONE DENSITY - OCT. 1987
- ■ FOUNDATION SAND CONE DENSITY- OCT. 1987
- ⊡ FOUNDATION UNDISTURBED DENSITY- JUNE 1987
- ⊞ FOUNDATION UNDISTURBED DENSITY- OCT. 1956
- ⊙ REMOLDED DENSITIES USED FOR LAB. SHEAR TESTS, COMPOSITES A & B - JAN. 1988

SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

LABORATORY COMPACTION
AND
FIELD DENSITY TEST RESULTS

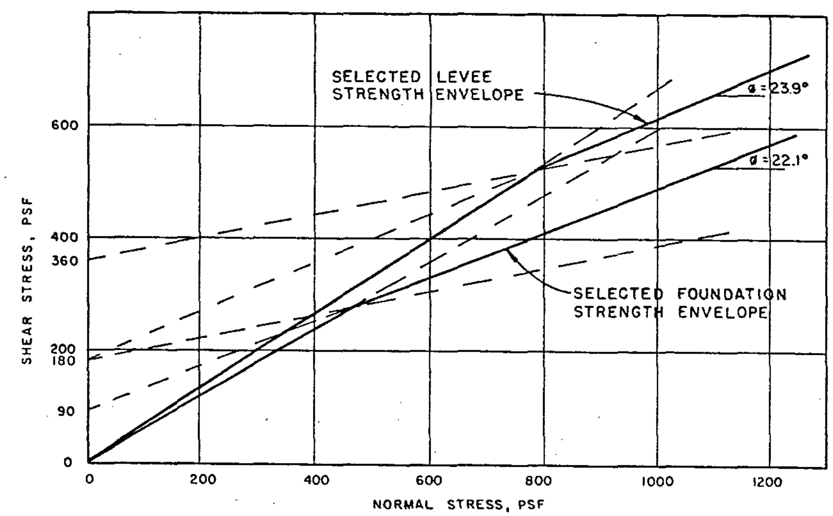
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

FIGURE 3

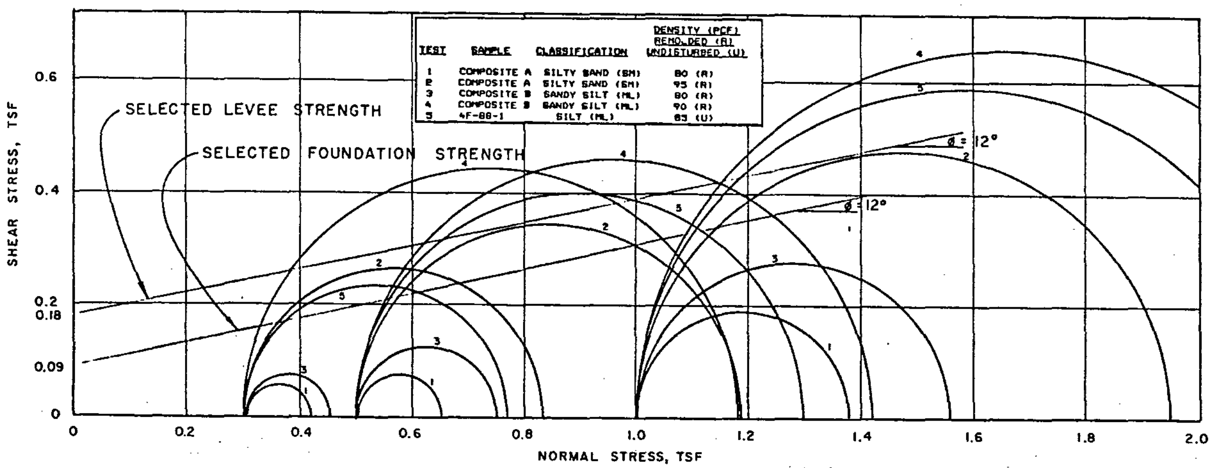


TEST	SAMPLE	CLASSIFICATION	DENSITY (PCF) REMOLDED (R) UNDISTURBED (U)	SHEAR STRENGTH (DEGREES)
1	COMPOSITE A	SILTY SAND (SM)	110 (R)	41
2	COMPOSITE A	SILTY SAND (SM)	95 (R)	39
3	COMPOSITE A	SILTY SAND (SM)	80 (R)	37
4	COMPOSITE B	SANDY SILT (ML)	100 (R)	37
5	COMPOSITE B	SANDY SILT (ML)	90 (R)	36.5
6	COMPOSITE B	SANDY SILT (ML)	80 (R)	35.5
7	4F-88-2	LEAN CLAY (CL)	92 (U)	29.5
8	2B-2 (1956)	SILTY SAND (SM)	105 (R)	33
9	2B-6 (1956)	SILTY SAND (SM)	92 (R)	27
10	2F-2 (1956)	SANDY SILT (ML)	85 (U)	26

DIRECT SHEAR



STRENGTH ENVELOPES



TEST	SAMPLE	CLASSIFICATION	DENSITY (PCF) REMOLDED (R) UNDISTURBED (U)
1	COMPOSITE A	SILTY SAND (SM)	80 (R)
2	COMPOSITE A	SILTY SAND (SM)	95 (R)
3	COMPOSITE B	SANDY SILT (ML)	80 (R)
4	COMPOSITE B	SANDY SILT (ML)	90 (R)
5	4F-88-1	SILT (ML)	85 (U)

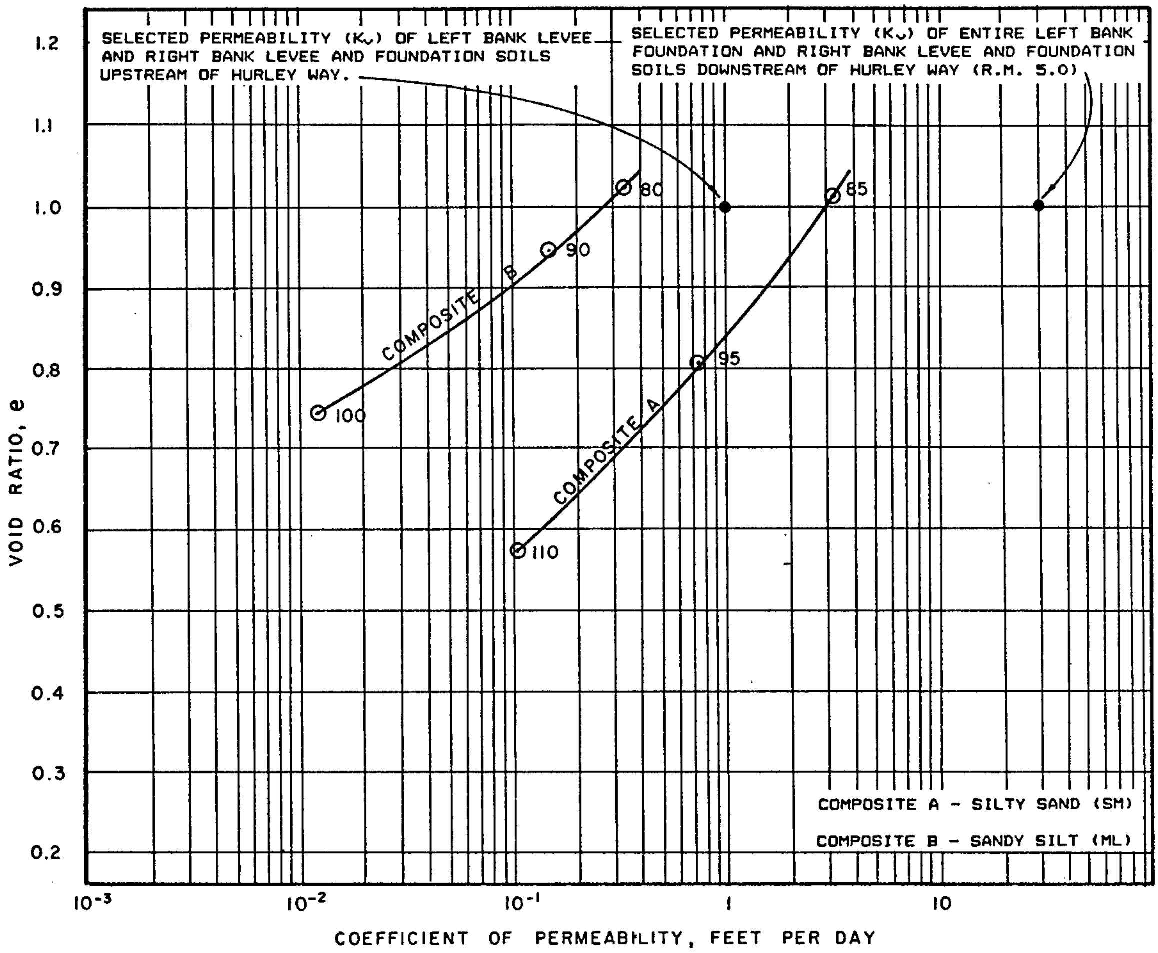
CONSOLIDATED - UNDRAINED SHEAR

SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

LABORATORY SHEAR STRENGTH TEST RESULTS AND SELECTED STRENGTHS

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

FIGURE 4



NOTES

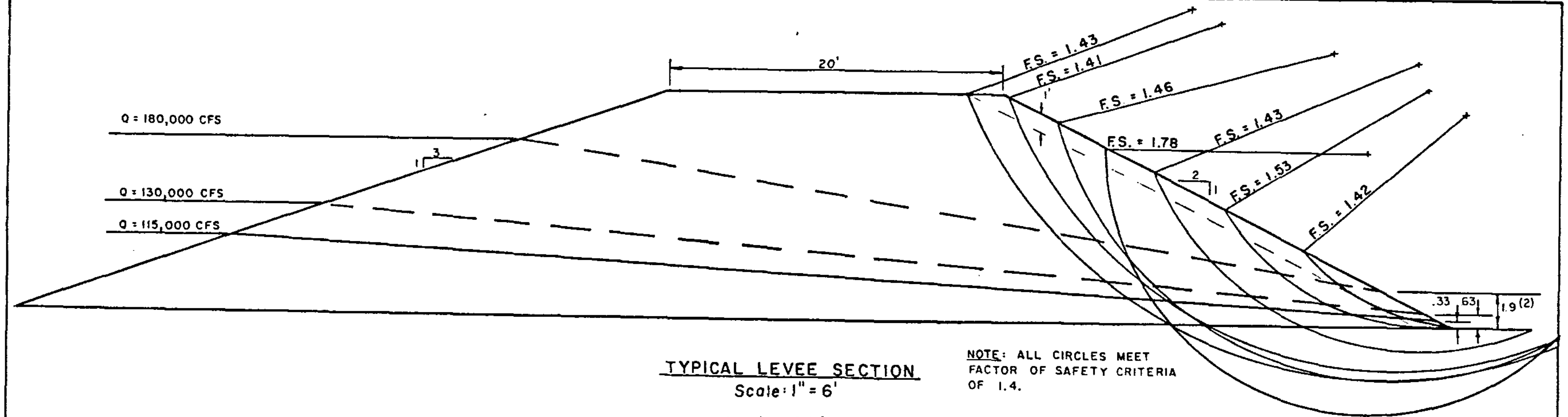
1. FIGURES BESIDE TEST POINTS INDICATE THE REMOLDED DENSITY PRIOR TO TEST.
2. ALL SPECIMENS TESTED WITH A CONFINING STRESS OF 1000 PSF.
3. SPECIMEN DATA: 4-INCH DIAM., 2-INCH HEIGHT.

SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

LABORATORY PERMEABILITY TEST RESULTS

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

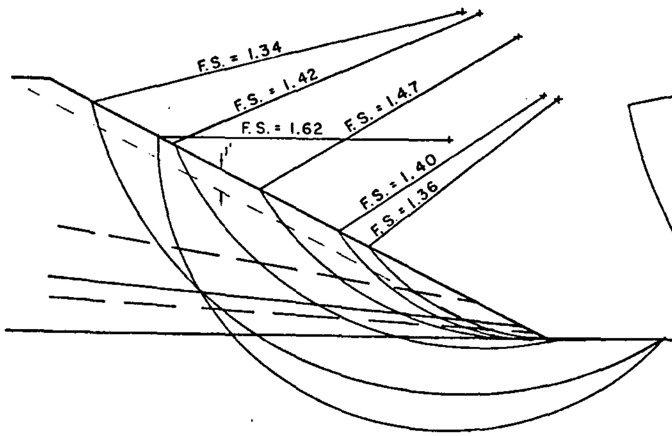
FIGURE 5



TYPICAL LEVEE SECTION
Scale: 1" = 6'

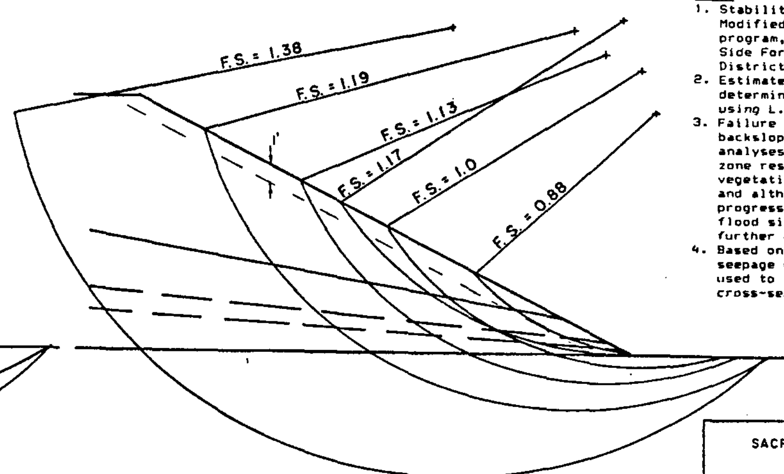
Q = 115,000 cfs

NOTE: ALL CIRCLES MEET FACTOR OF SAFETY CRITERIA OF 1.4.



Q = 130,000 cfs

NOTE: ALL CIRCLES BELOW THE UPPER 1-FOOT OF THE LEVEE SLOPE MEET THE FACTOR OF SAFETY CRITERIA OF 1.4.



Q = 180,000 cfs

NOTE: NONE OF THE ABOVE CIRCLES MEET MINIMUM FACTOR OF SAFETY CRITERIA.

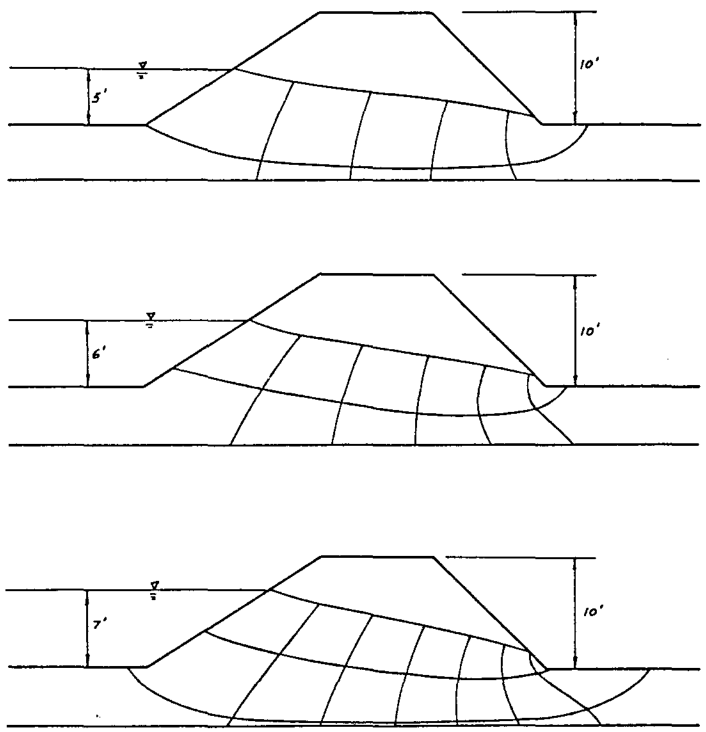
NOTES
1. Stability analyses perfomed using the Modified Swedish Method on computer program, Slip Circle Slope Stability with Side Forces developed by the St. Paul District, Corps of Engineers.
2. Estimated phreatic surface and exit point determined for a typical cross-section using L. Casagrande's procedure (Ref. 5).
3. Failure arcs in the upper foot of the levee backslope are not considered in the analyses because: Apparent cohesion in this zone resulting from existing grassy vegetation was not used in the analyses, and although shallow sloughing can be progressive in nature, levee patrols during flood situations can repair and prevent further damage.
4. Based on the stability analyses, a critical seepage water exit height of 0.60 feet was used to evaluate all of the levee cross-sections in this study.

SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

SLOPE STABILITY ANALYSES
TYPICAL LEVEE SECTION

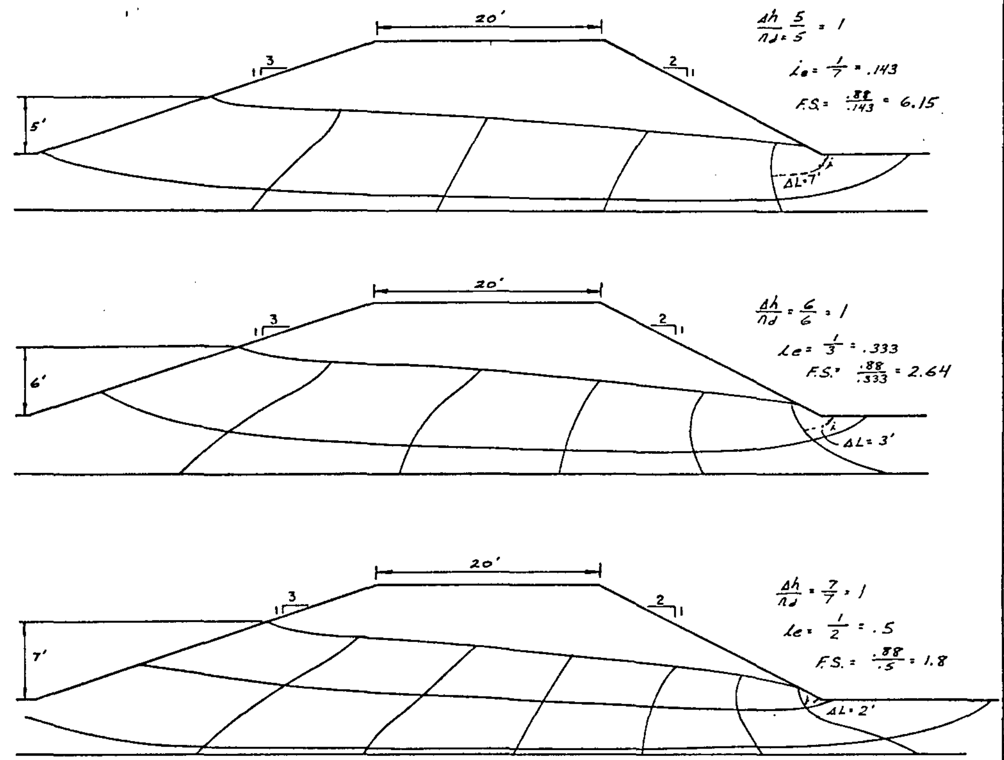
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

FIGURE 6



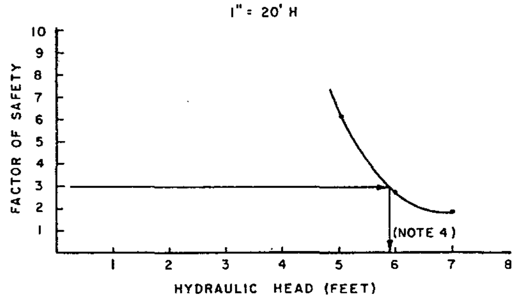
TRANSFORMED SECTIONS

1" = 10' V
1" = 20' H



TRUE SECTIONS

1" = 10'



NOTES

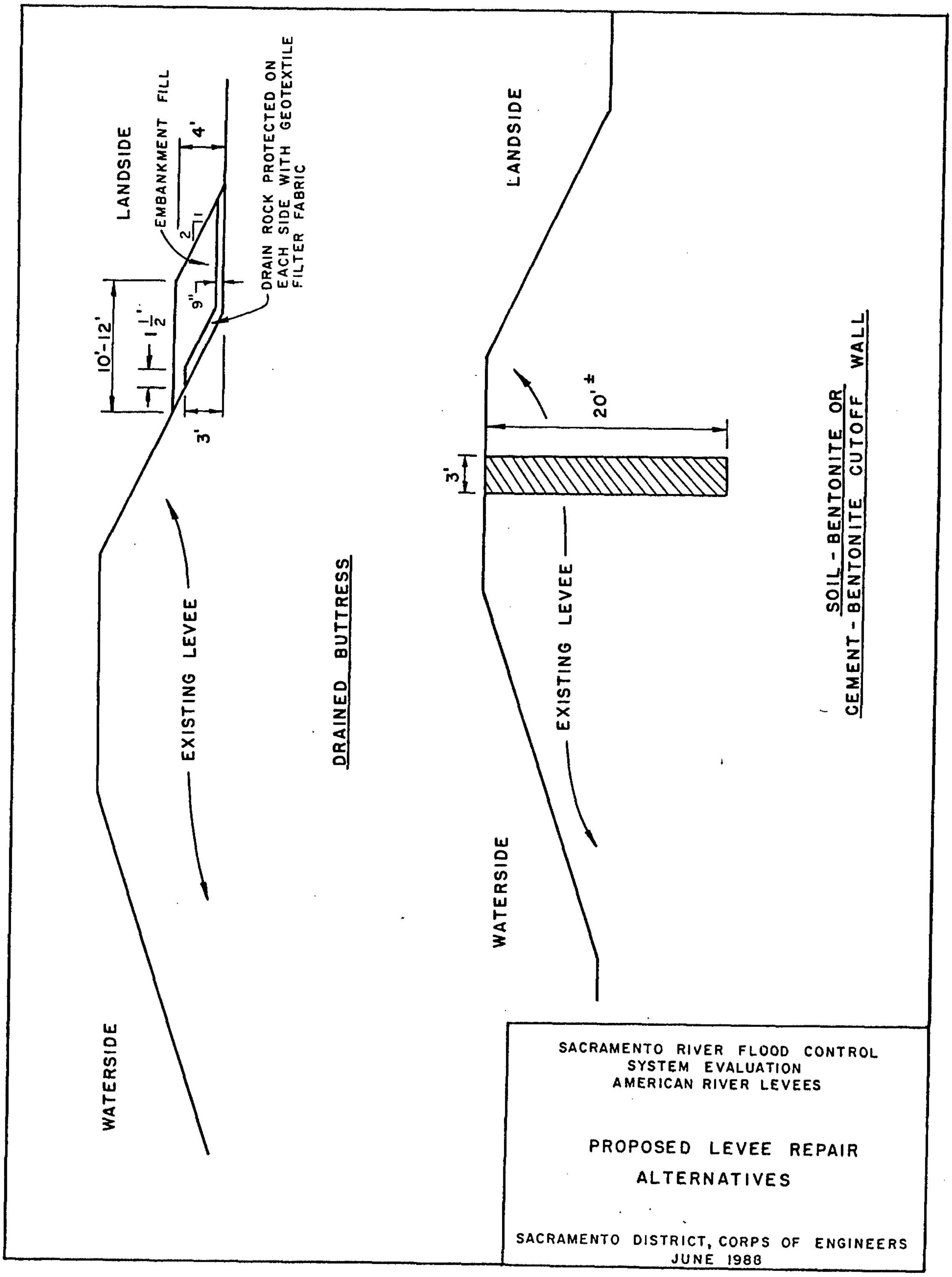
1. This flow net analysis determines the maximum hydraulic head on a typical levee section without exceeding a factor of safety of 3.0 against piping (internal erosion). Minor variations would result for levee sections of slightly different heights.
2. *Horizontal permeability assumed equal to four times the verical* permeability, ie. $k_h=4k_v$. k_v assumed to equal 30 feet/day.
3. Factor of Safety, $F.S.=i_c/i_e$
   where: i_c= critical exit gradient= $(G_s-1)/1+e)$
   $= (2.76-1)/(1+1) = 0.88$
   i_e= exit gradient near levee toe
4. A value of 6 feet was selected for the maximum allowable hydraulic head.

SACRAMENTO RIVER FLOOD CONTROL SYSTEM EVALUATION
AMERICAN RIVER LEVEES

FLOW NET ANALYSIS

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

FIGURE 7



SACRAMENTO RIVER FLOOD CONTROL
SYSTEM EVALUATION
AMERICAN RIVER LEVEES

PROPOSED LEVEE REPAIR
ALTERNATIVES

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1988

FIGURE 8

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 3

GEOTECHNICAL BASIS OF DESIGN
LEVEE DESIGN REQUIREMENTS
FOR
AMERICAN RIVER

AUGUST 1989

PREPARED BY

SOIL DESIGN SECTION
GEOTECHNICAL BRANCH

GEOTECHNICAL BASIS OF DESIGN
LEVEE DESIGN REQUIREMENTS
FOR
AMERICAN RIVER

TABLE OF CONTENTS

GEOTECHNICAL BASIS OF DESIGN
LEVEE DESIGN REQUIREMENTS
FOR
AMERICAN RIVER

BACKGROUND

Following the record flows on the American River in February 1986 re-evaluation of the American River flood control system was requested by the city and county of Sacramento. These culminated in the following reports:

a. Special Study on Lower American River, California - dated March 1987 by the Sacramento District for US Bureau to Reclamation and California Department of Water Resources.

b. Office Report, Levee Stability, Sacramento River Flood Control System Evaluation, American River Levees - dated July 1988.

c. Reconnaissance Report, - American River Watershed Investigation, California - dated January 1989.

Based on the findings in these studies, local interests opposed to new upstream storage have requested a study to upgrade the levees to pass floods by increasing the downstream capacity in the lower American River.

SCOPE OF WORK

Current design flow is 115,000 cfs and proposed objective flows are 130,000 cfs and 180,000 cfs. To accommodate these flows, raising portions of the levees will be required. Design freeboard requirements for these objective releases are discussed in Chapter 1 of Appendix N. It was determined that the levee reaches shown on Plate 1 on the lower American River should be studied. In conjunction with the study, the need for stone protection was to be coordinated with the hydraulic design engineers. This analysis is given in Chapter 4 of this Appendix. In order to estimate the cost to upgrade the levees to safely pass the objective flows, it was requested the following information be provided:

a. Levee and foundation soil types by reach.
b. Levee reaches which require stabilization to accommodate increased flood flows.
c. Type of stabilization to include berms, toe drains or cutoff walls.
d. Toe drain, berm, and cutoff wall design.
e. Enlarged levee section design to provide adequate freeboard.
f. Basis of design.

SITE CONDITIONS

On 18 July 1989, soil design and hydraulic design personnel took a boat ride up the American River from its confluence with Sacramento River to Watt Avenue bridge to examine soils in the river bank, to evaluate river bank and levee slope conditions and to determine areas requiring slope protection.

The banks of the American River are predominately silty sand and sandy silt with lesser amounts of clayey silt. The banks that stand steeper than 1V on 2H are predominately silt or sands reinforced with vegetation either roots or grass (See Plates 2 and 3 for typical banks). Closer examination of the banks revealed that the materials were often deposited in layers from 6 to 24 inches thick. The levees appear to have been predominately constructed on the old river terrace. The levees are constructed of similar silty sand and sandy silt observed in the river terrace; however, the levee fill is predominately homogeneous rather than layered.

On 26 and 27 July 1989 other District personnel examined the American River levees to: (1) determine existing site conditions, (2) develop possible levee alignments on the south side of the river east of Mayhew Drain, (3) identify physical constraints to levee construction and (4) verify soil types.

The overall condition of the levees was judged to be very good. Detailed recorded information regarding levee conditions is retained by Central Valley Section. The following general comments summarize field observations. Foot traffic has created areas that are denuded and therefore more susceptible to erosion. The sands in the levees are highly erosive and some footpaths have developed into erosion rills. These rills should be repaired before they develop into ravines. The landside slope and toe of the levee in the Riverpark area (River Mile 5 to River Mile 7) has been encroached upon with residential development. This encroachment makes emergency flood fighting more difficult. The alignment of the levee is difficult to determine at the gravel pit located near River Mile 3 west of the sanitary landfill. However, the ground has been built up in this area so that flooding should not be a problem. This area needs more study and future borrow pit operations must be monitored. The freeboard on the levee along the Mayhew Drain has been reduced due to construction of a relatively low bridge. The private levee east of Mayhew Drain appears marginally lower than the main river levees and are not well maintained. This levee has been encroached upon in many areas, and development has been permitted on the waterside of the levee. The north levees are in better condition. In general they are not as large with no encroachment and are well maintained. However, east of the Northeast sewage sanitation plant the site conditions differ. Here, the levee has been encroached upon both on the landside and waterside slopes.

A levee alignment for the new south levee east of Mayhew Drain is shown on Plate 1.

Many constraints or obstacles exist along the left levee. These consist primarily of fences, utility lines and landscaping. The constraints extend the entire length of the levee. In many places bike paths or access roads will need to be moved. On the right side, west of the Northeast sewage plant very few constraints exist. There are many pumping plants to tie the proposed subsurface drainage features. There are some access roads and bike paths that will require realignment. In addition there are many large features including bridges, roads, water treatment plants and sewage treatment plants that may need portions relocated.

During the 26 and 27 July field trips, the material in the levees were found to be silty sand and sandy silt as found in earlier studies.

SOIL TYPES

a. Foundation - The American River drains a portion of the central Sierra Nevada Mountains. The present channel is at least 10,000 years old. The river alluvium is a product of (1) the tectonic rise of the Sierra Nevada Mountains, fall of the central valley, and associated metamorphism and volcanics; (2) the changes in climate to include periods of glaciation and deposition; (3) weathering and erosion of the mountains and (4) man's influence that include hydraulic mining and the construction of upstream dams. The levee's foundation is primarily an old terrace consisting primarily of recent alluvial sand, silt and clay of granitic origin. Below the Business 80 Highway bridge, the foundation materials are influenced by alluvial deposits of the Sacramento River and minor tributaries of North East Sacramento as well as the American River.

As illustrated on Plate 4 the foundation consists primarily of fine sandy silt and clayey silt of low plasticity downstream of the Business Highway 80 bridge. The fines content is generally over 70 percent. On the right bank upstream of Business 80 bridge to the Guy A West bridge, the foundation consists of both silty sand and sandy silt, whereas upstream of the Guy A West bridge the foundation consists predominately of sandy silt. However, the fines portion, are between 50 and 70 percent and the materials are non-plastic. On the left bank upstream of the Business 80 bridge to Mayhew Drain the foundation consists predominately of sandy silt. Similarly, the fines range between 50 and 70 percent and are non-plastic. Upstream of Mayhew Drain, subsurface explorations have not been made. Soil survey maps indicate the foundation materials are similar to downstream soils but contain less fines.

b. Levees - The levees were constructed with nearby or adjacent waterside borrow. See Plate 5 for material composition. The soil materials in the left levee downstream of Watt Avenue consist predominately of silty sand, whereas upstream of Watt Avenue the levees consist of sandy silt. The right bank consists primarily of sandy silt with shorter reaches of silty sand.

STABILIZATION AND SEEPAGE CONTROL FEATURES

The July 1988 Office Report determined areas that require stabilization by both drains and berms. Both toe drains and internal cutoff walls are required. Where physical constraints along the left levee toe do not allow sufficient area to install a toe drain a cutoff wall will be required. Some reaches require a berm and toe drain whereas other reaches require a toe drain only. A typical toe drain is shown on Plate 6; a berm and toe drain is shown on Plate 7, and a cutoff wall is illustrated on Plate 8. Reaches where a berm and toe drain or internal cutoff wall are required are shown on Plate 9. The toe drain must drain to an existing pumping plant or other outfall drainage feature. These were identified during the field investigations.

LEVEE ENLARGEMENT

Previous studies indicate that enlarging the levees an additional four feet in height will not significantly change the stability. However, it is important that the levee fill be constructed properly. Plate 10 designates those reaches which must have landside or waterside fill placement to increase levee height. Plate 11 has been included to illustrate the steps to obtain a stable levee with landside construction, while Plate 12 illustrates waterside construction.

LEVEE AND BANK STONE PROTECTION REQUIREMENTS

Requirements for erosion protection for the increased objective releases are discussed in Chapter 4 of this Appendix. Plate 13 indicates those reaches that are now protected with stone and those reaches that require erosion protection as described in Chapter 4. A few other reaches were identified as actively eroding.

REFERENCES

1. As-constructed drawings, "Sacramento River Flood Control Project, Proposed Levee Enlargement, South Levee of the American River, 16th Street Bridge to Mayhew, "Drawing file no. 1-4-362, December 1947.

2. As-constructed drawings, "Sacramento River Flood Control Project, American River, Natomas Canal, Arcade Creek and Linda Creek, Levee Construction," Drawing file no. 1-4-392, December 1954.

3. As-constructed drawings, "American River Flood Control Project, Fair Grounds to Carmichael Bluffs, Levee Construction and Bank Protection," Drawing file no. 1-4-415, February 1957.

4. Design Memorandum No. 1 - American River Project, California, American River Levee General Design, 25 June 1956.

5. Wahler Associates, Investigation for Levee Repair, American River Flood Control District (ARFCD) and the California Department of Water Resources, Maintenance Area 9 (DWR M.A. 9), Sacramento River, Sacramento County, California," July 1986.

6. Special study on the Lower American River, California Department of the Army, Sacramento District, Corps of Engineers, Sacramento, California, March 1987.

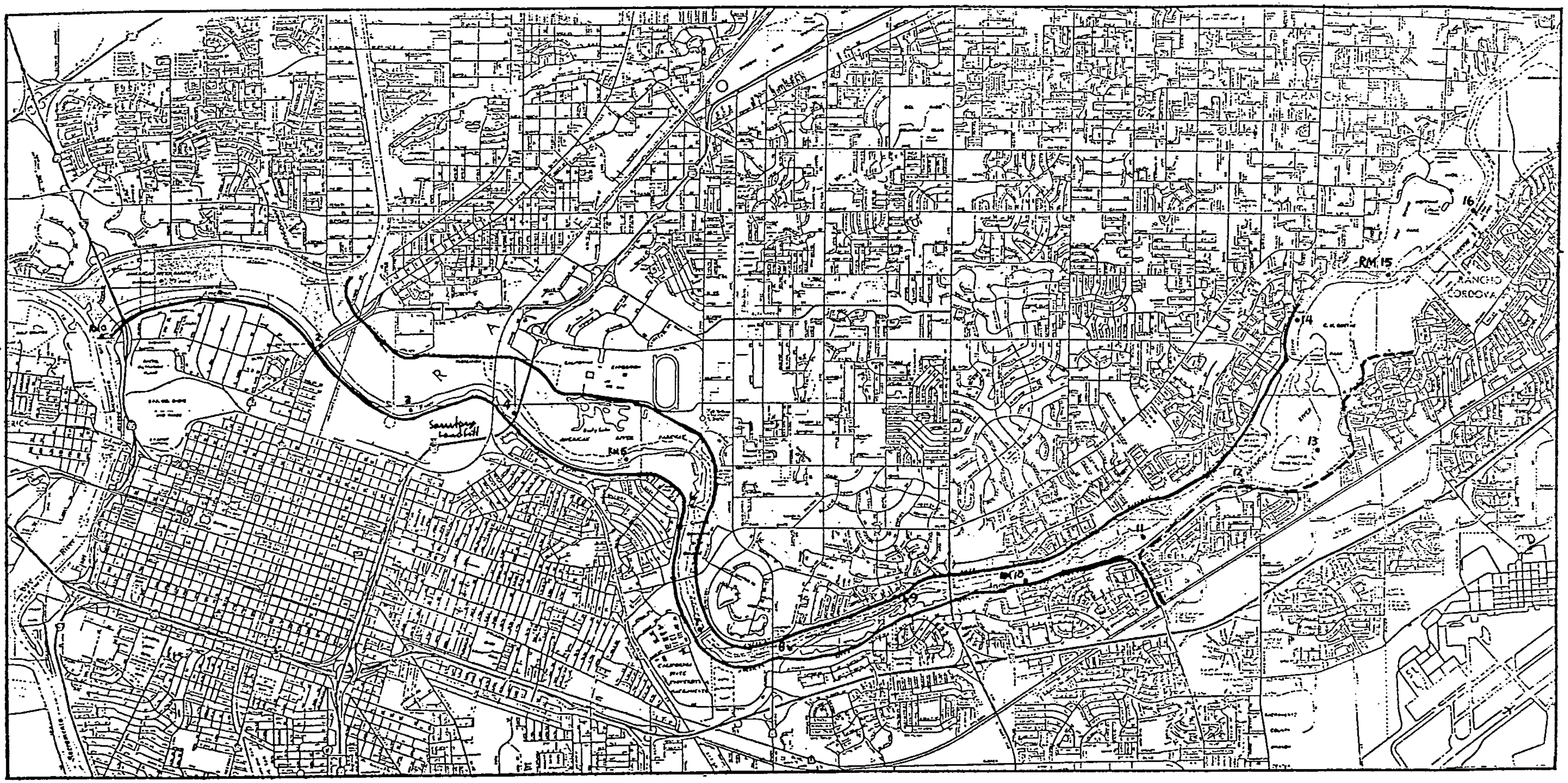
7. Geotechnical Reconnaissance Report, American River Watershed, California, Soil Design Section, Geotechnical Branch, U.S. Army Engineer District, Sacramento Corps of Engineers, May 1987.

8. Wahler Associates, "Levee Investigation, American River Flood Control District (ARFCD) and Maintenance Area 10 and 11 (MA 10 and 11), Dry Creek, Arcade Creek, Natomas East Main Drainage Canal, American River, and Sacramento River, Sacramento County, California," September 1987.

9. Soil Survey, Sacramento Area California, United States Department of Agriculture, Soil Conservation Service in cooperation with the University of California Agricultural Experiment Station, Series 1941, No. 11 Issued August 1954.

10. Geologic History of Middle California - Arthur D. Howard, 1979.

11. Landform - Soil Relationships in Northern Sacramento County, California, Roy J. Shlemond, 1967.



LEGEND

——— LEVEE ALIGNMENT

– – – PROPOSED LEVEE ALIGNMENT

RM 5 RIVER MILE 5

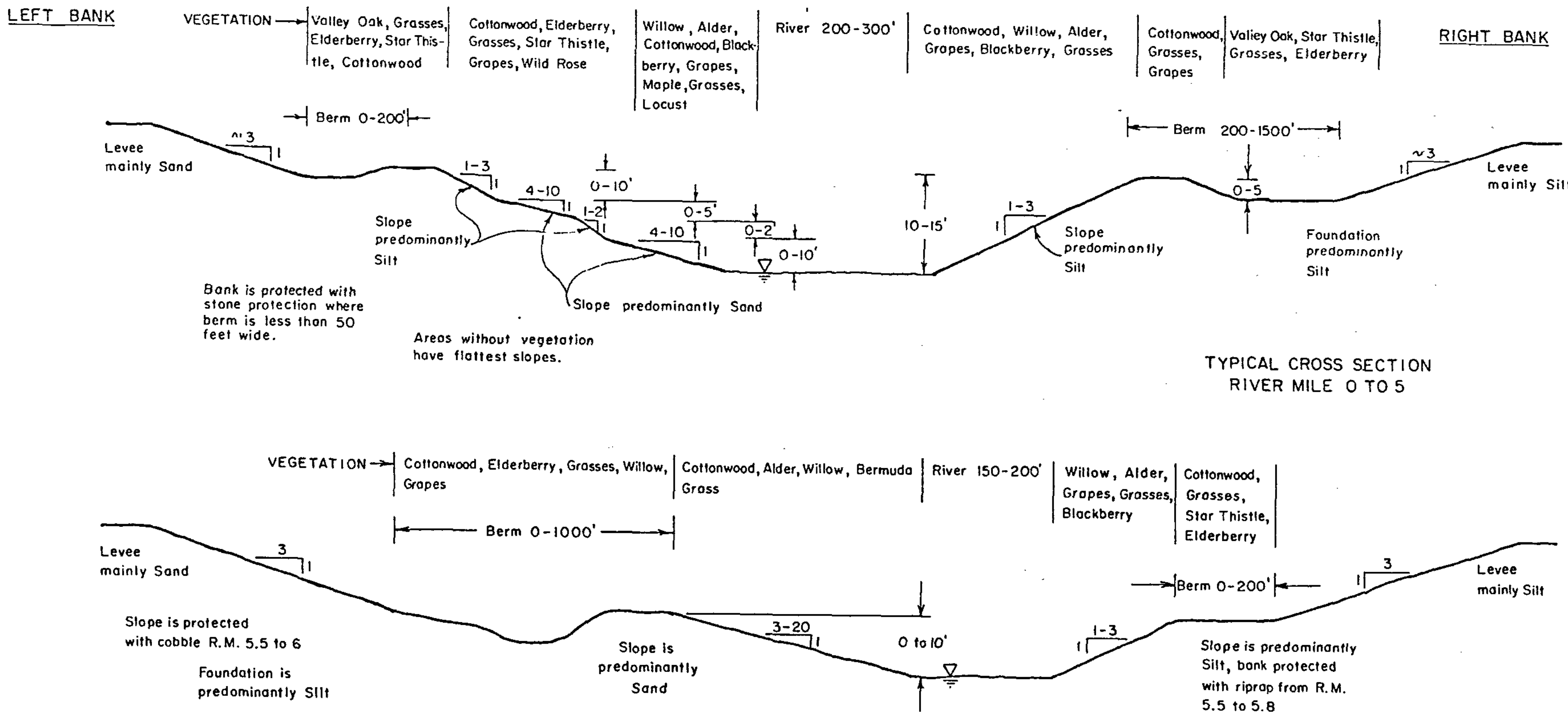
N

GRAPHIC SCALE

1"=4000' 0 2000' 4000' 8000' 12000'

STUDY LEVEES
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE

PLATE 1

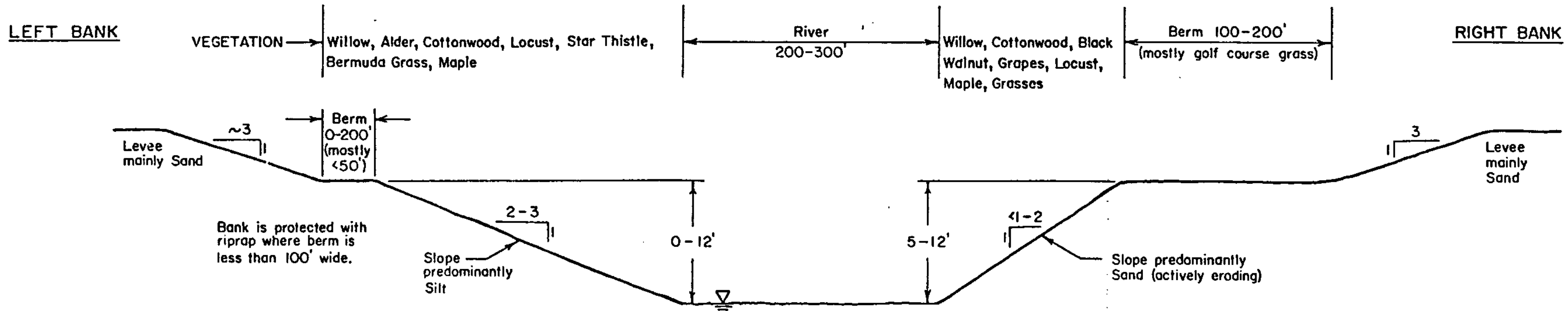


TYPICAL CROSS SECTION
RIVER MILE 0 TO 5

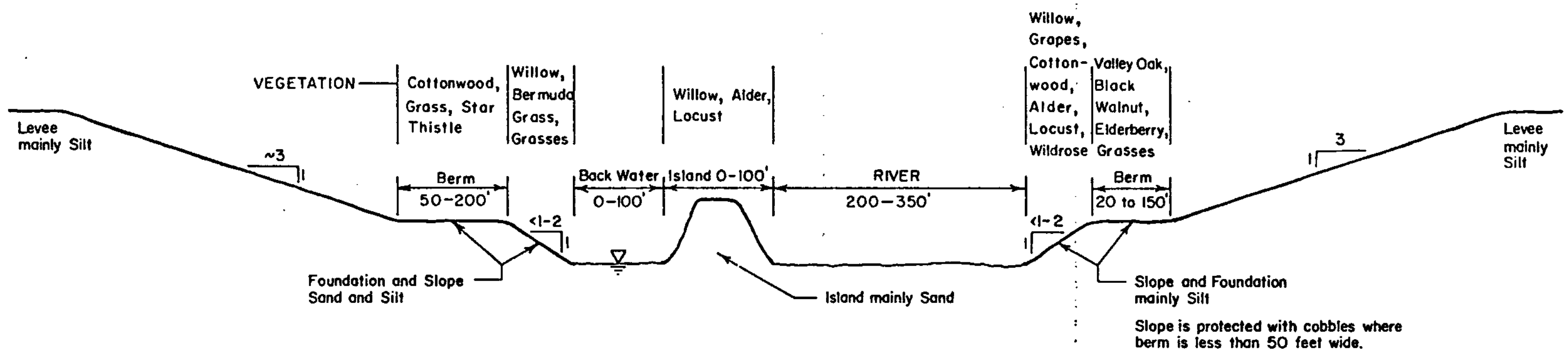
TYPICAL CROSS SECTION
RIVER MILE 5 TO 6

AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE

PLATE 2



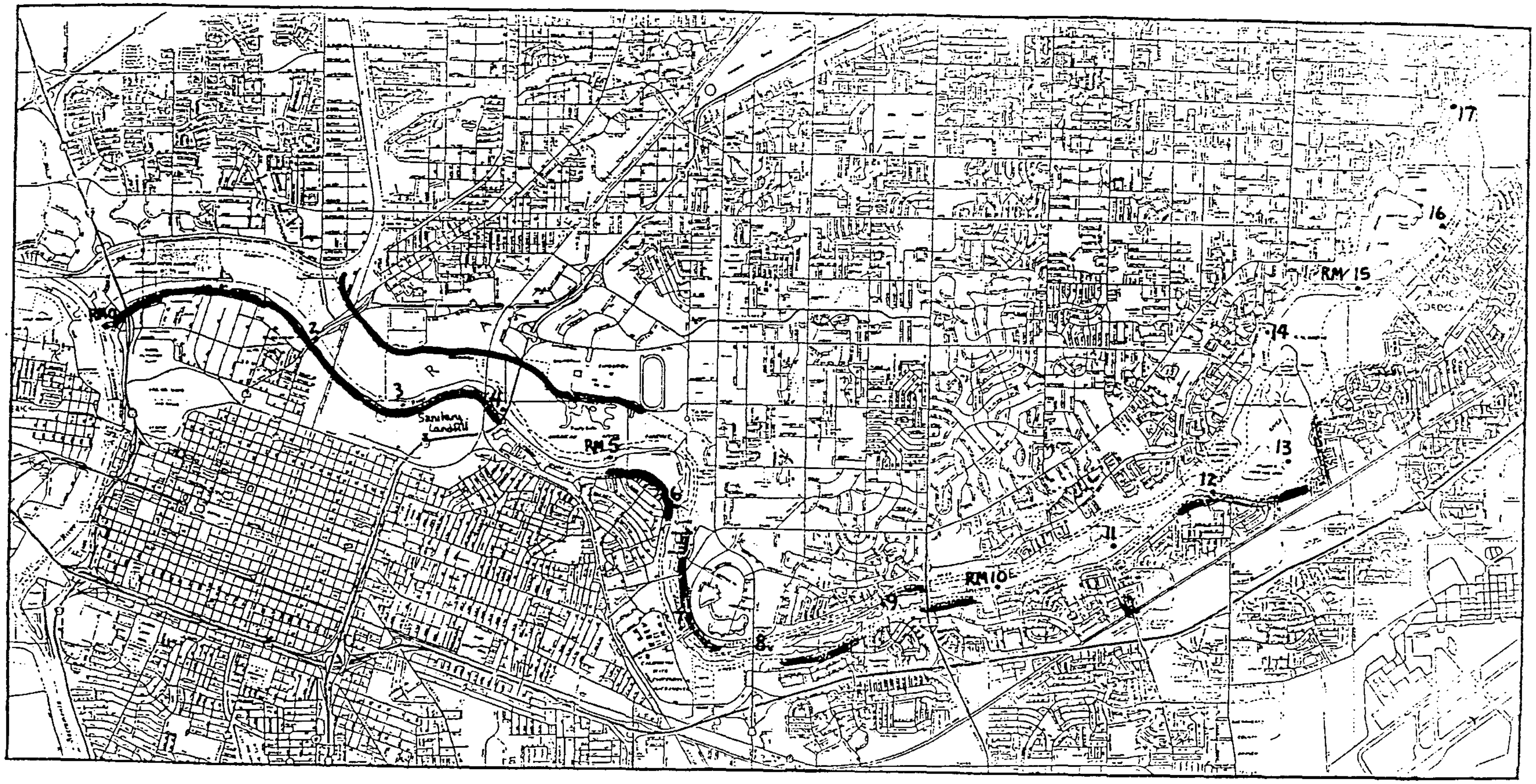
TYPICAL CROSS SECTION
RIVER MILE 6 TO 8



TYPICAL CROSS SECTION
RIVER MILE 8 TO 9

AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE

PLATE 3



LEGEND

- GARBAGE FOUND IN FOUNDATION
- SANDY SILT, SILT AND CLAYEY SILT
- SANDY SILT AND SILTY SAND
- SILTY SAND AND SANDY SILT

RM 5 RIVER MILE 5

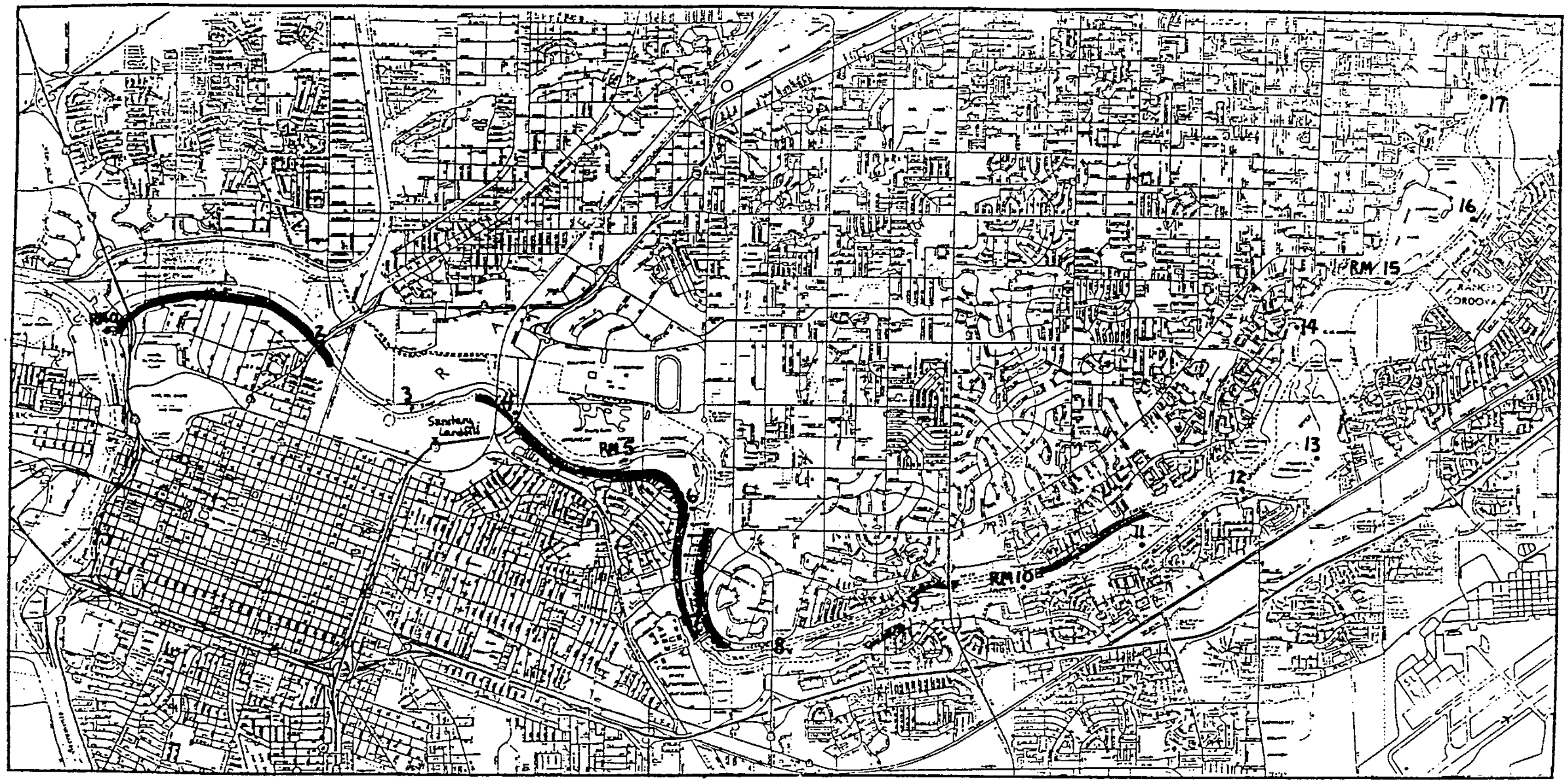
N

GRAPHIC SCALE

1"=4000' 0 2000' 4000' 8000' 12000'

FOUNDATION MATERIALS
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIV

PLATE 4



LEGEND

SANDY SILT AND SILTY SAND

SILTY SAND AND SANDY SILT

RM 5 RIVER MILE 5

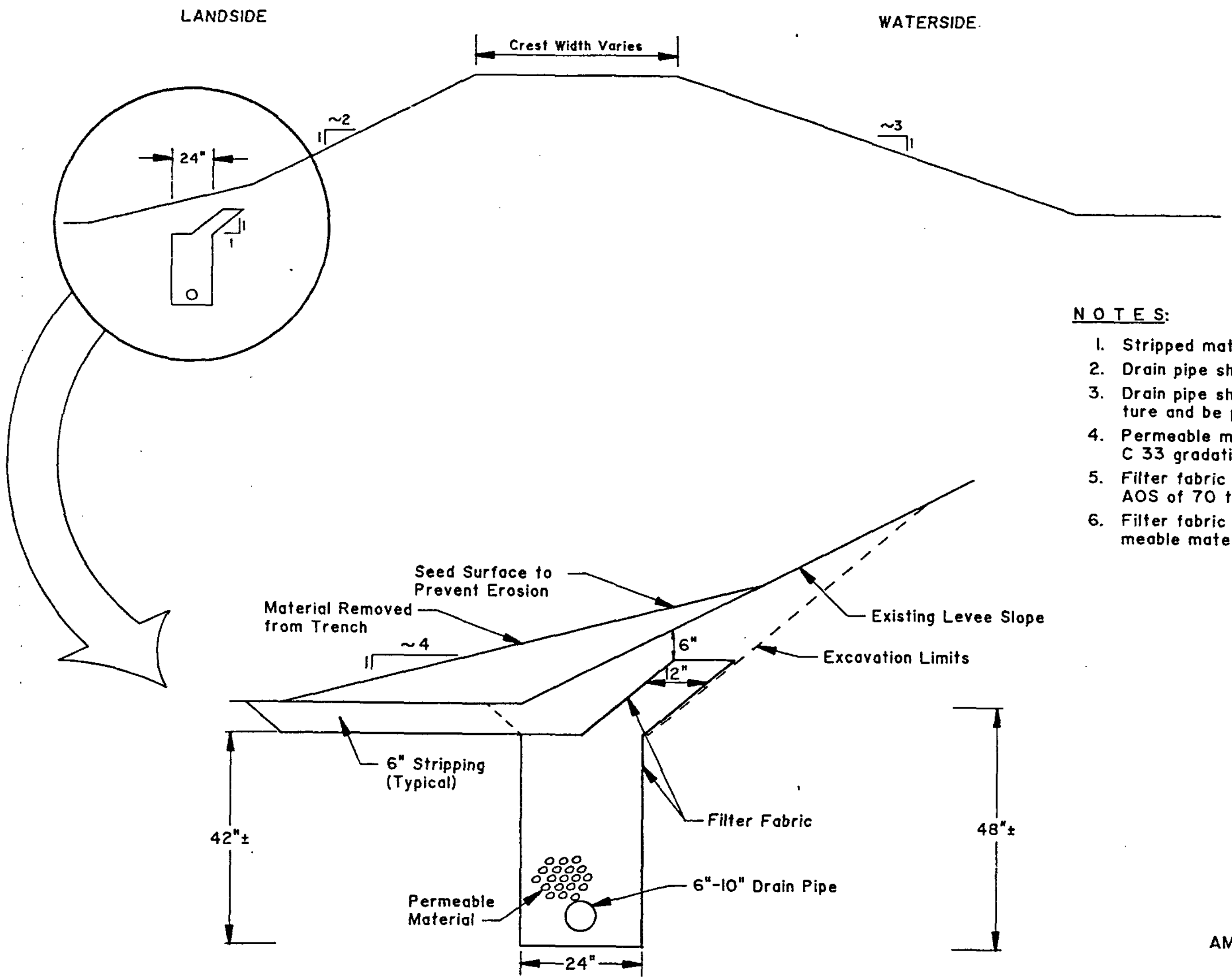
N

GRAPHIC SCALE

1"=4000' 0 2000' 4000' 8000' 12000'

LEVEE MATERIALS
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATI

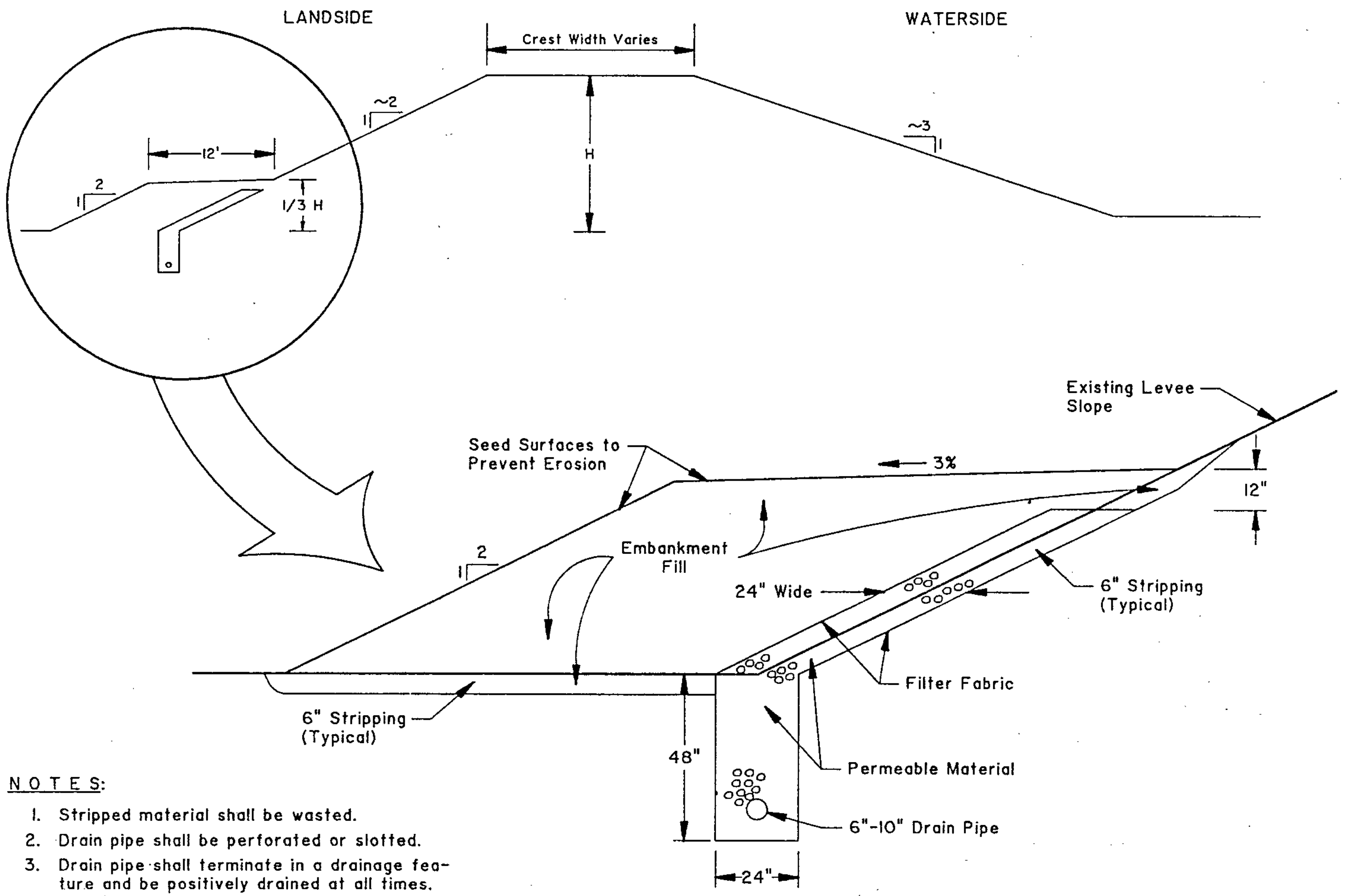
PLATE 5



N O T E S:

1. Stripped material shall be wasted.
2. Drain pipe shall be perforated or slotted.
3. Drain pipe shall terminate in a drainage feature and be positively drained at all times.
4. Permeable material shall conform to ASTM C 33 gradation 57 or 67.
5. Filter fabric shall be a woven fabric with an AOS of 70 to 120.
6. Filter fabric shall completely surround permeable material.

TOE DRAIN
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE



N O T E S:

1. Stripped material shall be wasted.
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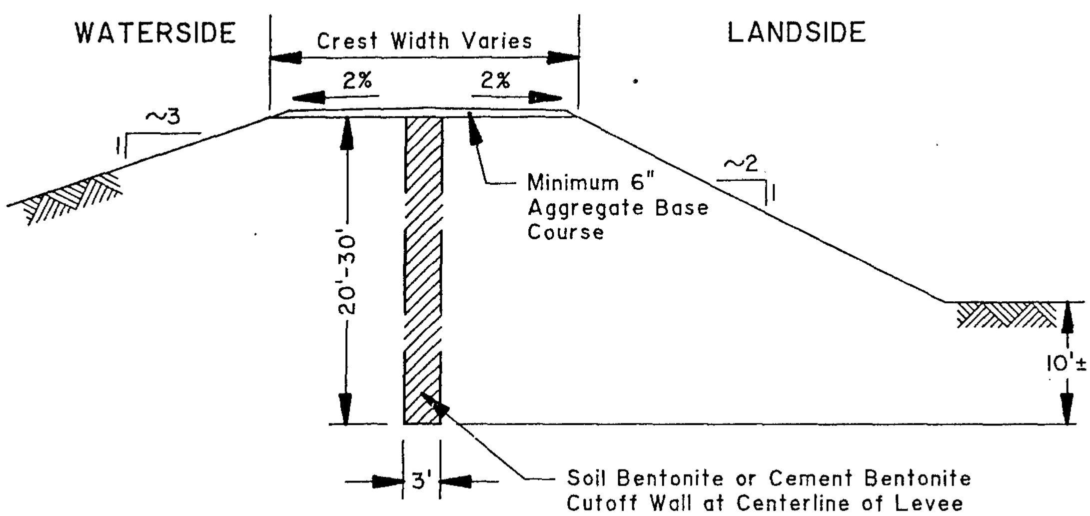
3. Drain pipe shall terminate in a drainage feature and be positively drained at all times.
4. Permeable material shall conform to ASTM C 33 gradation 57 or 67.
5. Filter fabric shall be a woven fabric with an AOS of 70 to 120.
6. Filter fabric shall completely surround permeable material.

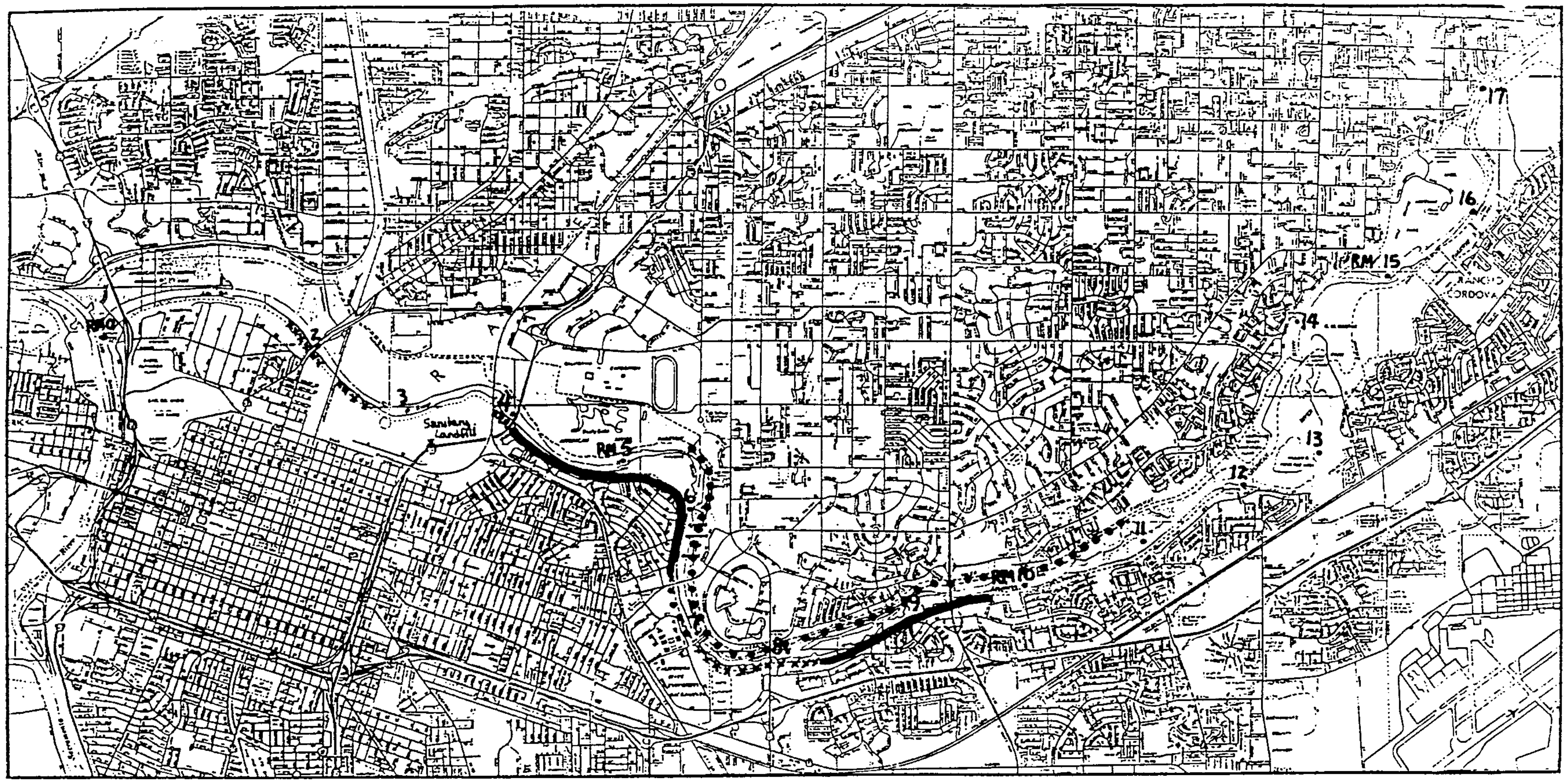
BERM WITH TOE DRAIN

AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE



TYPICAL SECTION

CUTOFF WALL
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE



LEGEND

xxxxx LANDSIDE BERM WITH TOE DRAIN

SLURRY WALL

••••• TOE DRAIN

RM 5 RIVER MILE 5

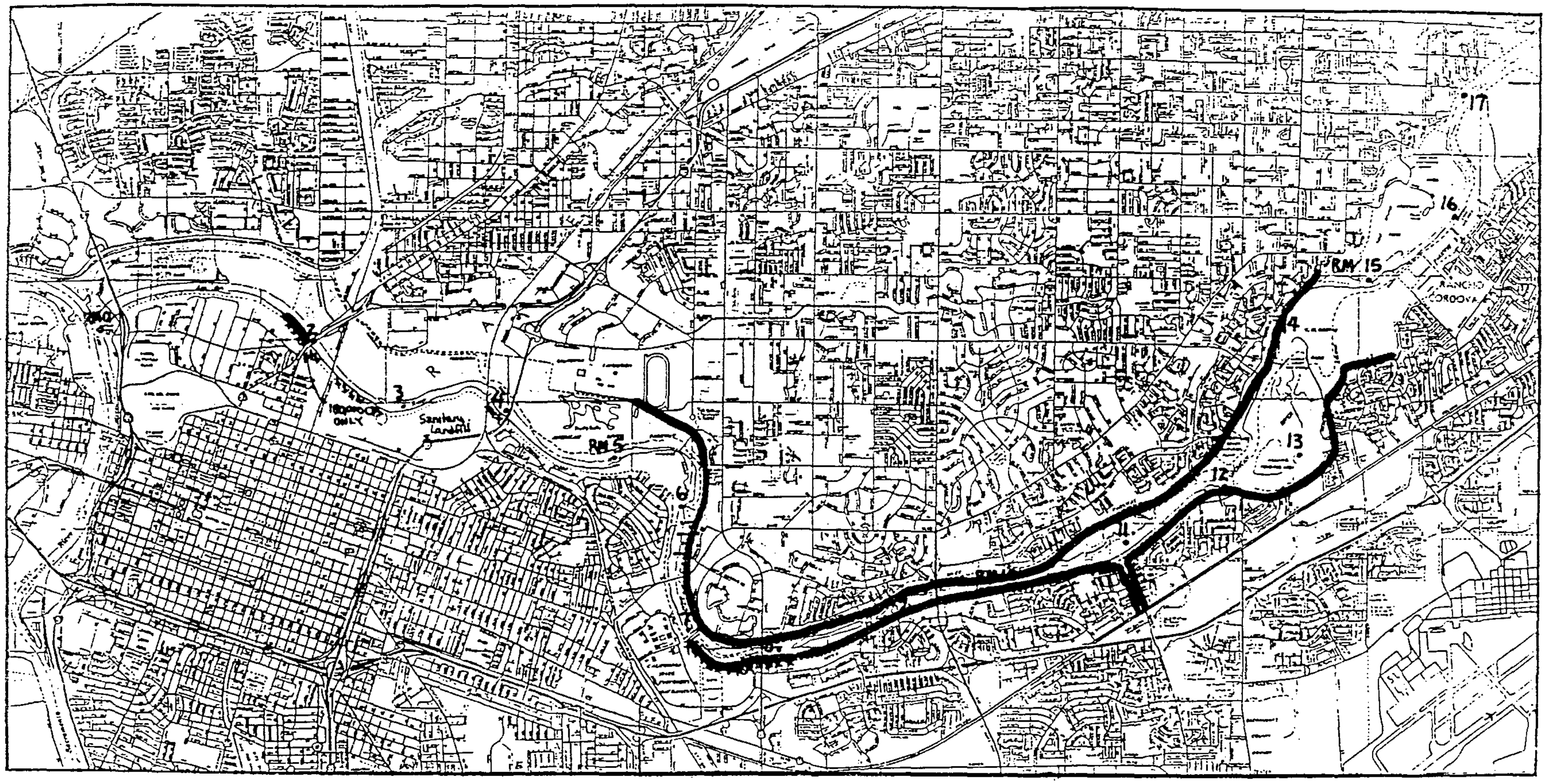
N

GRAPHIC SCALE

1" = 4000' 0 2000' 4000' 8000' 12000'

DRAINAGE FEATURES
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE

PLATE 9



LEGEND

xxx xxx LANDSIDE BERM WITH DRAIN

FILL LANDSIDE

FILL WATERSIDE

RM 5 RIVER MILE 5

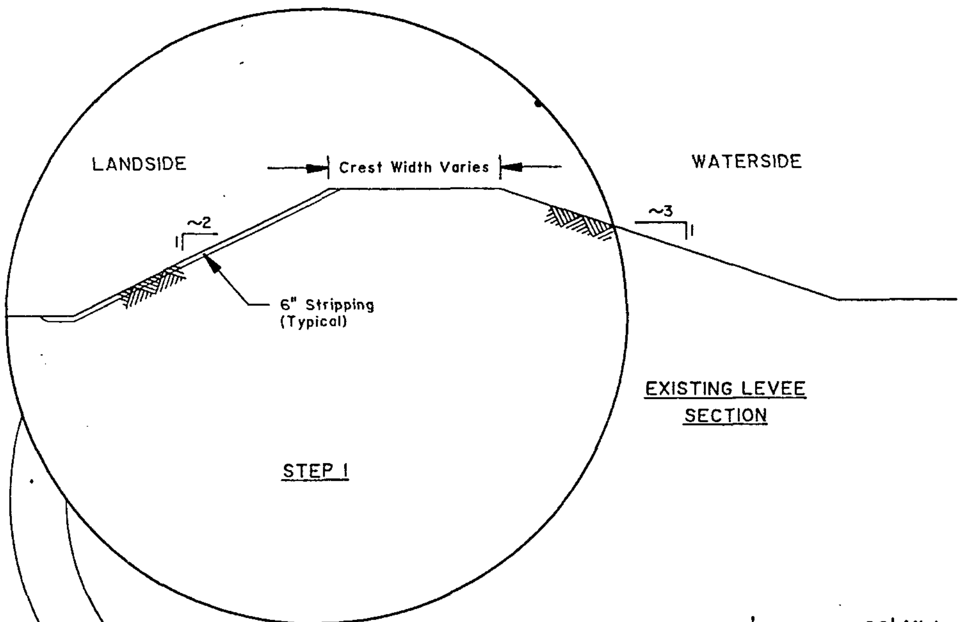
N

GRAPHIC SCALE

1"=4000' 0 2000' 4000' 8000' 12000'

FILL AREAS
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE

PLATE 10



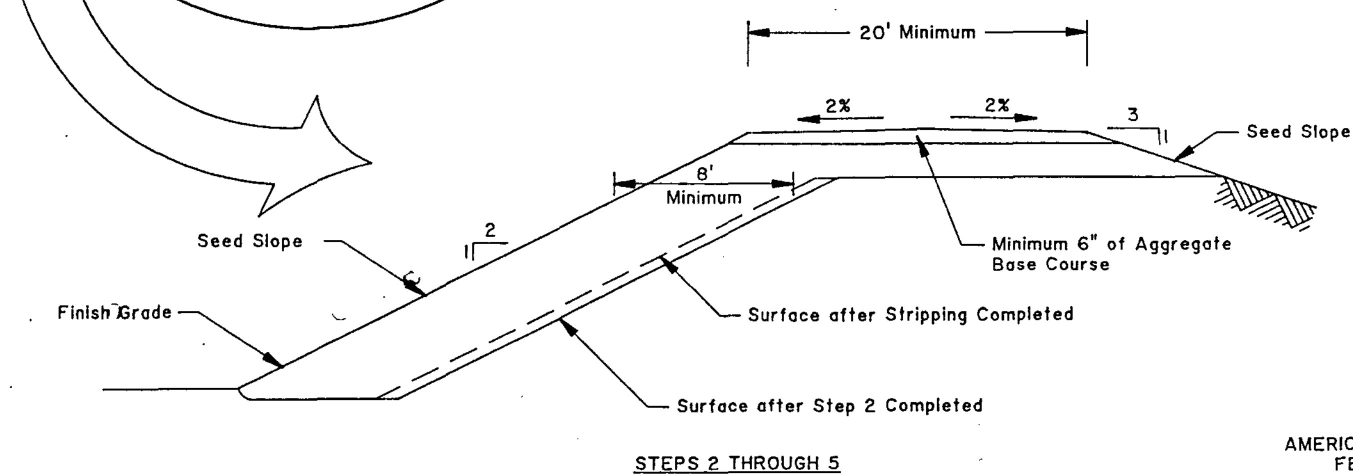
STEP 1: Strip 6" (waste material).

STEP 2: Remove material, if necessary, from Landside slope so that fill can be placed on a minimum 8-foot wide bench (if satisfactory material is removed, it may be used for embankment fill).

STEP 3: Place embankment material to lines and grades indicated. Materials shall be compacted to a minimum 95% of maximum density determined by ASTM D 698 at ±3% of optimum moisture content.

STEP 4: Place a minimum of 6 inches of aggregate base course. Aggregate shall be compacted to a minimum of 100% of maximum density determined by ASTM D 698 at ±3% of optimum moisture content (crest width must be a minimum of 20 feet when construction is completed).

STEP 5: Seed and establish grasses on new slopes for erosion protection.



FILL LANDSIDE
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE

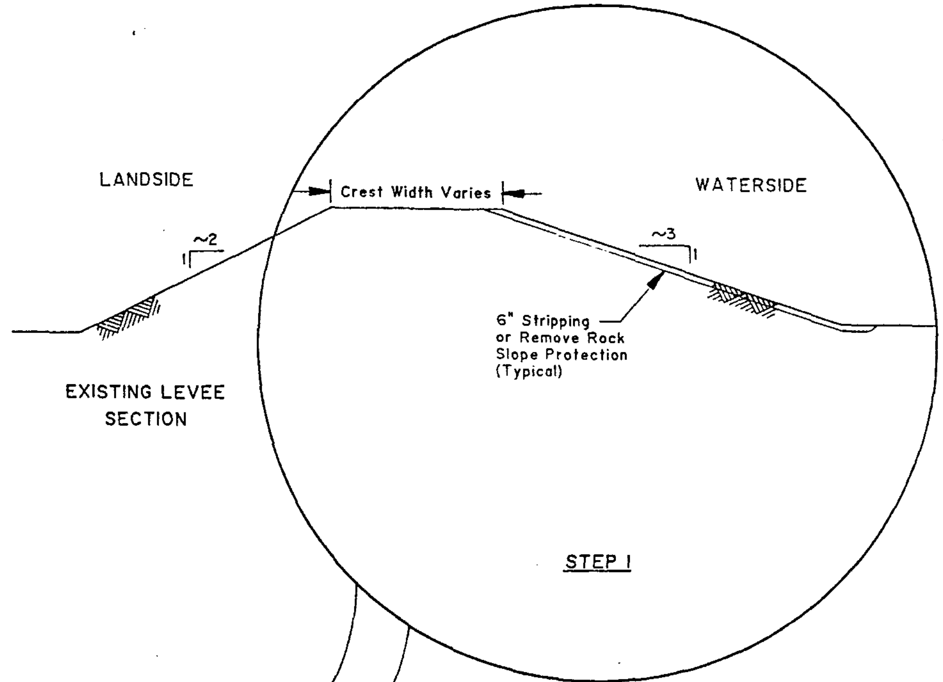
STEP 1: Strip 6" (waste material). If rock slope protection is on the slope, remove and save.

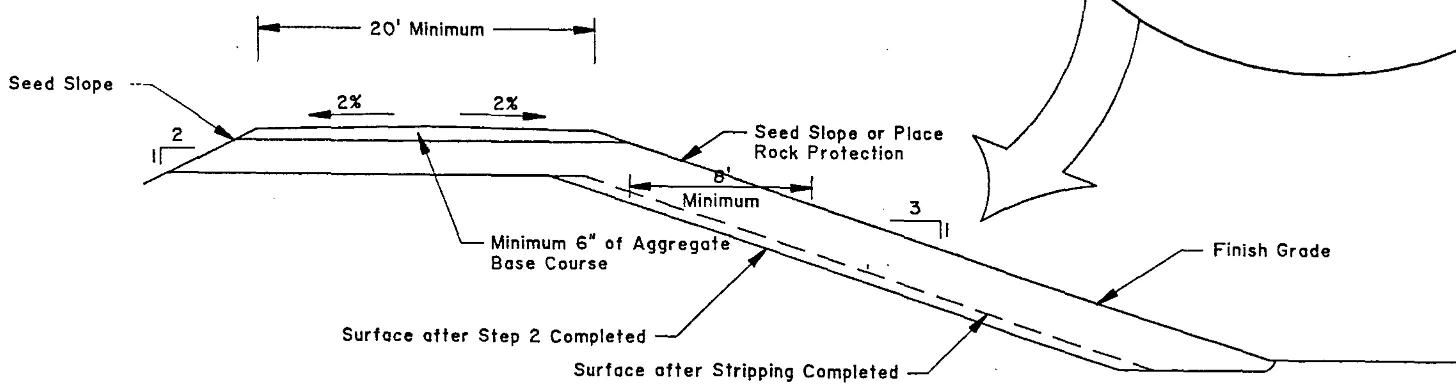
STEP 2: Remove material, if necessary, from Waterside slope so that fill can be placed on a minimum 8-foot wide bench (if satisfactory material is removed, it may be used for embankment fill).

STEP 3: Place embankment material to lines and grades indicated. Materials shall be compacted to a minimum 95% of maximum density determined by ASTM D 698 at ±3% of optimum moisture content.

STEP 4: Place a minimum of 6 inches of aggregate base course. Aggregate shall be compacted to a minimum of 100% of maximum density determined by ASTM D 698 at ±3% of optimum moisture content (crest width must be a minimum of 20 feet when construction is completed).

STEP 5: Seed and establish grasses on new slopes for erosion protection. Where rock slope protection was removed, return to slope.

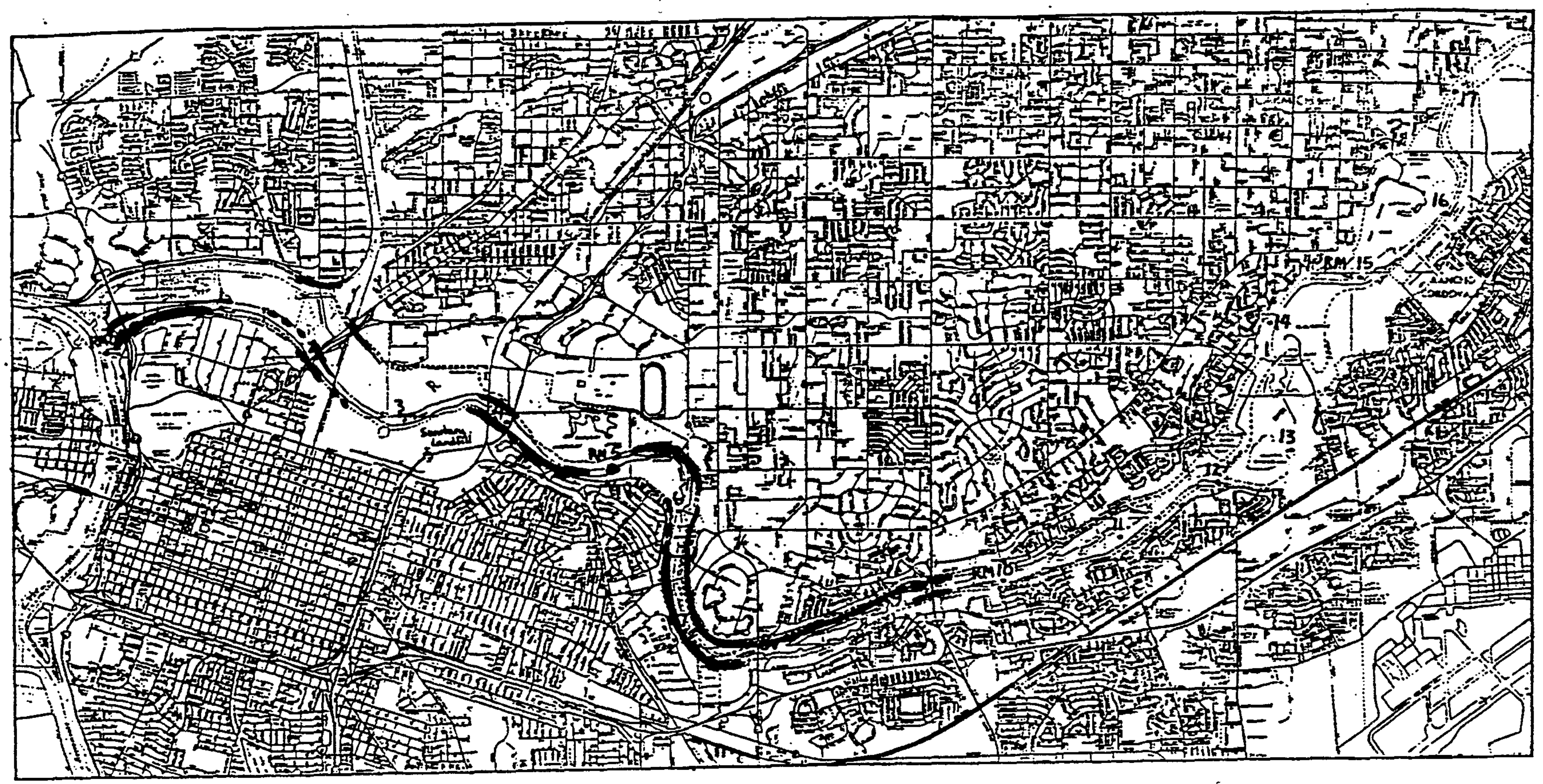




STEPS 2 THROUGH 5

FILL WATERSIDE
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE

PLATE 12



LEGEND

••• EXISTING STONE PROTECTION

━━ OTHER AREAS REQUIRING STONE PROTECTION

RM 5 RIVER MILE 5

N

GRAPHIC SCALE

1" = 4000'

STONE PROTECTION
AMERICAN RIVER WATERSHED
FEASIBILITY STUDY
AUGUST 1989
LOWER AMERICAN RIVER ALTERNATIVE

PLATE 13

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 4

BASIS OF DESIGN
EROSION PROTECTION REQUIREMENTS
FOR
AMERICAN RIVER

AUGUST 1989

PREPARED BY

HYDRAULIC DESIGN SECTION
DESIGN BRANCH

LOWER AMERICAN RIVER
130,000 AND 180,000 CFS OBJECTIVE RELEASE MEASURES
EROSION PROTECTION

TABLE OF CONTENTS

TABLE

FIGURES

LOWER AMERICAN RIVER
130,000 AND 180,000 CFS OBJECTIVE RELEASE MEASURES
EROSION PROTECTION

INTRODUCTION

This report is prepared to provide an analysis of the need, location, and design of rock slope protection along the American River from its confluence with the Sacramento River (River Mile (RM) 0) to Goethe Park (RM 15) for an increase in the objective flow from Folsom Dam to 130,000 and 180,000 cfs.

PRIOR STUDIES

The following documents were reviewed:

a) Special Study on Lower American River, California, dated March 1987
b) Reconnaissance Report, American River Watershed Investigation, California, dated January 1989
c) Office Report, Levee Stability, Sacramento River Flood Control System Evaluation, American River Levees, dated July 1988
d) Geotechnical Reconnaissance Report, American River Watershed, California, prepared by Soil Design Section, dated May 1987.

A review of the HEC 2 Data Set from the Sacramento Flood Insurance Study for FEMA, developed in 1988 was used to obtain cross sections and design velocities for the study reach.

FIELD INSPECTIONS

On-site inspections were made of the study reach levees and channel in July 1989. An inspection by boat was made by Hydraulic and Soil Design engineers on July 18, 1989. Surface flow at the time of the inspection was 6,500 cfs. In general, the banks are composed of mainly sandy silt and silty sand. There were some areas that consisted of clay. The lower reaches were heavily vegetated with large trees and thick brush as well as wide berms. Other areas had no berm and very little vegetation with little or no bank protection.

A ground reconnaissance of the study reach was also conducted on

July 24, 1989. Surface flow at the time of the inspection was 5,000 cfs. Heavy vegetation was observed on many parts of the river. There were areas with a wide berm and heavy vegetation and other areas where the incised channel abutted the project levees. Downstream of Goethe Park the project levee on the left bank ends. Small private levees are in place from Goethe Park upstream to Nimbus Dam. The right bank has homes built right to the river edge. Bank erosion was noted on the left bank from RM 0.5 to RM 4. The toe of the left bank near California State University, Sacramento is also eroding. There was evidence that several bridge abutments and areas of levee had been rocked since the 1986 flood.

ASSUMPTIONS AND CRITERIA

Water surface elevations and design velocities for the study area were developed in 1988, by the HEC 2 program for the Sacramento Area Flood Insurance Study. The model was calibrated using the 1986 high water marks and cross sections taken in 1987. Cross section data in the study reach was obtained from the 1988 FEMA Study for the Sacramento area. Bank protection requirements for the American River were determined using a discharge of 130,000 and 180,000 cfs. A 20% reduction in boundary roughness conditions in the FEMA HEC 2 data deck was assumed. The average channel velocities ranged from 3 to 16 feet per second.

Channel stone riprap protection was designed in accordance with EM 1110-2-1601, "Hydraulic Design of Flood Control Channels" and ETL 1110-2-120, Incl 1, "Additional Guidance for Riprap Protection" assuming a unit weight of 165 lbs per cubic foot. For the American River, 12 "typical cross-sections" were used to represent the study area. The analysis was done using depths averaging from 15 to 30 feet, representing the approximate toe depths. Table M-4-1 lists the location, design velocity, layer thickness as well as the length and whether the levee, channel or bridge abutment has been recommended for revetment. Average velocities in the reach from RM 0.0 to RM 6.0 ranged from 4.5 to 7.5 ft/sec (130,000 cfs) and 5.5 to 10 ft/sec (180,000 cfs). The computed minimum layer thickness is 12 inches, however, due to the uncertainties of turbulence during the design flows, 15 inches is recommended. The suggested minimum weight of rock, W50 min, is 34 lbs for areas identified in the reach from RM 0.0 to RM 6.0 for both design flows.

In the reach from RM 6.0 to RM 8.0, the velocities are expected to exceed 12 ft/sec (130,000 cfs) and 15 ft/sec (180,000). The computed minimum layer thickness is 21 inches (130,000 cfs) and 27 inches (180,000 cfs). These thicknesses are recommended for areas identified in this reach. The W50 min to be used in this reach is 93 lbs (130,000 cfs) and 197 lbs (180,000 cfs). The reach from RM 8.0 to Goethe Park (RM 14.0) has expected

velocities from 5 to 9.5 ft/sec (130,000 cfs) and 6.0 to 11 ft/sec (180,000). The computed minimum layer thickness of 12 inches (130,000 cfs) and 18 inches (180,000). The recommended layer thickness for the 130,000 cfs is 15 inches and 18 inches for 180,000 cfs for the areas identified in this reach. The W50 min to be used in this reach varies from 34 to 58 lbs.

RECOMMENDATIONS

After review of the study area and written materials, the following are the recommendations for riprap along the American River from the mouth to Goethe Park. All bridge abutments will need to have slope protection at both levee and channel locations. Although the study limit is Goethe Park, all bridge abutments upstream to Nimbus Dam should also be protected. All recommended sites from the May 1987 Geotechnical report for riprap have been included as areas for riprap. Channel bottom stability under increased design flow conditions was not reviewed at this time. However, based on observation of materials along the study reach and computed velocities, it is recommended that channel stabilizers be constructed downstream of each bridge, as shown on Plate 40 of EM 1110-2-1601. There appears to be scour holes developing downstream from the Watt Avenue Bridge and between 12th Street and Union Pacific Railroad Bridges that are candidates for channel stabilizers at the present time.

Type of sites to be revetted are channel bank only, levee embankment only, channel bank and levee, and bridge abutments. For channel bank only, the potential scour depths during design events are unknown. A rock toe should extend a minimum of ten feet below the channel thalweg. Provision of "roll-back" rock at the top of the rock site on the berm should be made to protect the rock site from overtopping flows (Figure 1). An example of this is upstream of the I-5 Bridge at Discovery Park. The levee embankment only option is specified at many locations. In these reaches, the levee is presently setback a sufficient distance from the main channel bank so that only protection of the levee embankment will be required. A toe is provided for this type of site to protect the critical interface between the levee embankment and berm (Figure 2). An example of this is the area near Cal Expo. There are some locations where the channel bank is sufficiently close to the levee embankment to warrant protection of the channel bank and levee in a continuous slope or the channel bank, berm and levee embankment (Figure 3). An example of this is the area through the CSU, Sacramento reach. For the purposes of the Feasibility Study, it should be assumed that all bridge abutments in the study reach shall be protected. If feasible, the typical rock section would be similar to that for the levee embankment.

ADDITIONAL STUDIES

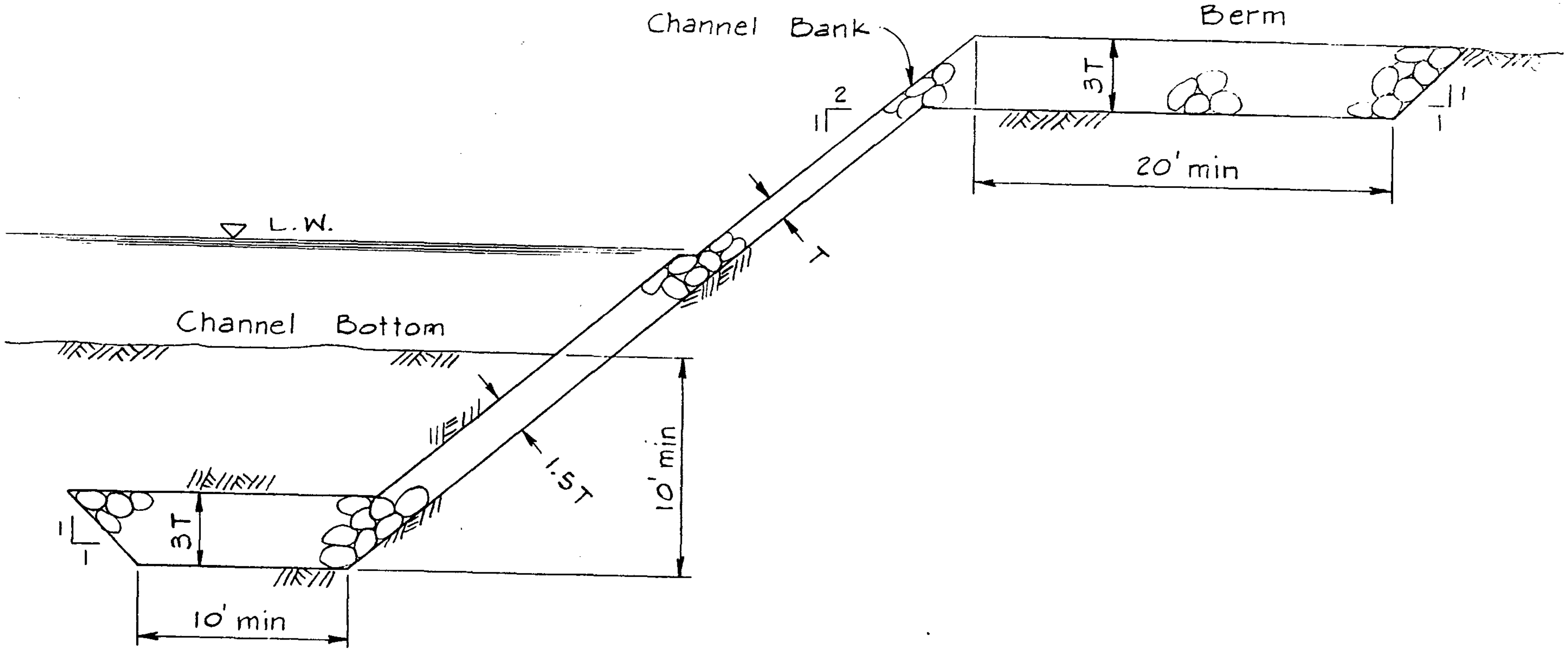
Should this measure become part of the selected plan, there will be a need for additional studies during Preconstruction Engineering and Design. The study area levees will need to be evaluated more thoroughly by a team composed of hydraulic design and soil design Engineers. The team will confirm the necessity for riprap by inspection and soil samples of the levees.

TABLE M-4-1

RECOMMENDATIONS FOR RIPRAP THICKNESS

RIVER MILE (APPROX)	LOCATION	LT/RT BANK	LENGTH (FT)	TYPE SITE	DESIGN VEL (FT/SEC) 130,000/ 180,000 cfs	WEIGHT (LBS) min W50 130,000/ 180,000 cfs	THICKNESS (IN) 130,000/ 180,000 cfs
0.0	Sacto Riv Levee, American Riv Levee, Jibboom & I-5 Br (Geotech site #1)	LT	6000	L,C,B	7.0/9.4	34/34	15/15
0.2	Jibboom St & I-5 Br.	RT	1500	C,B	7.0/9.4	34/34	15/15
1.7	Northgate Bl across NEMDC	RT/LT	200	B,L	7.8/10.2	34/34	15/15
1.9	Levee at 12th St causeway	RT	400	B,L	7.8/10.2	34/34	15/15
1.9	U/S of 12th St Br, Bike, RR Br (Geotech site #2)	LT	3000	B,CB	7.8/10.2	34/34	15/15
2.0	12th St, Bike, RR Br	RT	2000	B,C	7.8/10.2	34/34	15/15
3.7	Union Pacific RR Xing	RT	400	B,C	4.7/5.6	34/34	15/15
3.9	Business I-80	RT	400	B,C	4.7/5.6	34/34	15/15
3.6	Misc Bridge Abutments	RT	500	B	4.7/5.6	34/34	15/15
3.8	RR Br to Bus. I-80	LT	1500	B,C	4.7/5.6	34/34	15/15
4.1	Bus I-80 to Paradise Beach	LT	6000	L	7.9/9.0	34/34	15/15
5.3	U/S of Cal Expo	RT	3000	L	7.3/8.6	34/34	15/15
6.4	D/S of H St Br to U/S of Guy West Br	RT	4000	L	12.4/15.1	93/197	21/27
6.4	D/S of H St Br to Sac State (Sac State to Guy West Geotech site #3)	LT	5000	CB	12.4/15.1	93/197	21/27
7.2	Guy West Br to Howe Ave Br	LT	4800	L,B	12.4/15.1	93/197	21/27
7.2	U/S Guy West Br to U/S Howe Ave Br (Geotech site #4)	RT	3500	L,B	12.4/15.1	93/197	21/27
8.1	U/S Howe Ave Br D/S Watt Ave Br (Geotech site #5)	RT	5500	L	9.6/11.0	34/58	15/18
9.3	Watt Ave Br	RT	200	B	5.1/5.9	34/34	15/15
9.3	Watt Ave Br	LT	200	B	5.1/5.9	34/34	15/15
9.4	U/S of Watt Ave (Geotech site #6)	LT	4000	L	9.6/11.0	34/58	15/18
	TOTAL CHANNEL BANK ONLY		11800				
	TOTAL LEVEE ONLY		37400				
	TOTAL CHANNEL BANK AND LEVEE		8000				
	TOTAL BRIDGE ABUTMENT		4300				

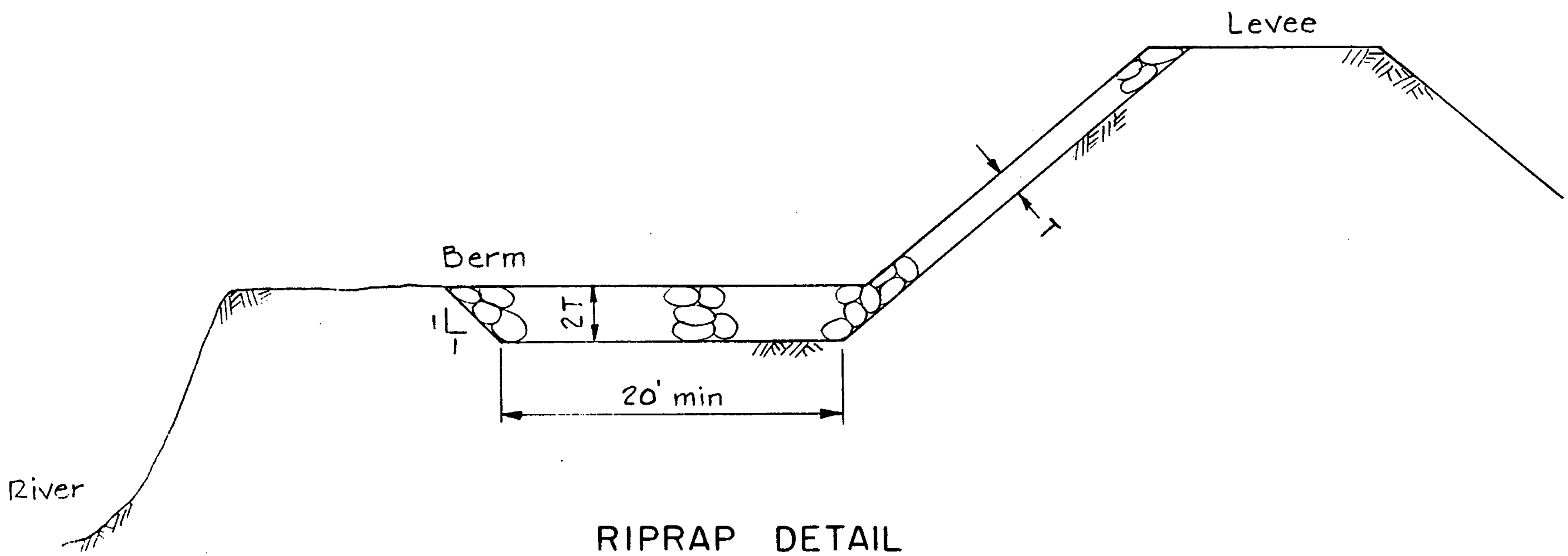
NOTES:
C = Channel Bank Only
L = Levee Only
CB = Channel Bank and Levee
B = Bridge Abutment
All stations represent the approximate start with distance going upsstream



RIPRAP DETAIL
NOT TO SCALE

LOWER AMERICAN RIVER
CHANNEL BANK
ONLY

FIGURE 1



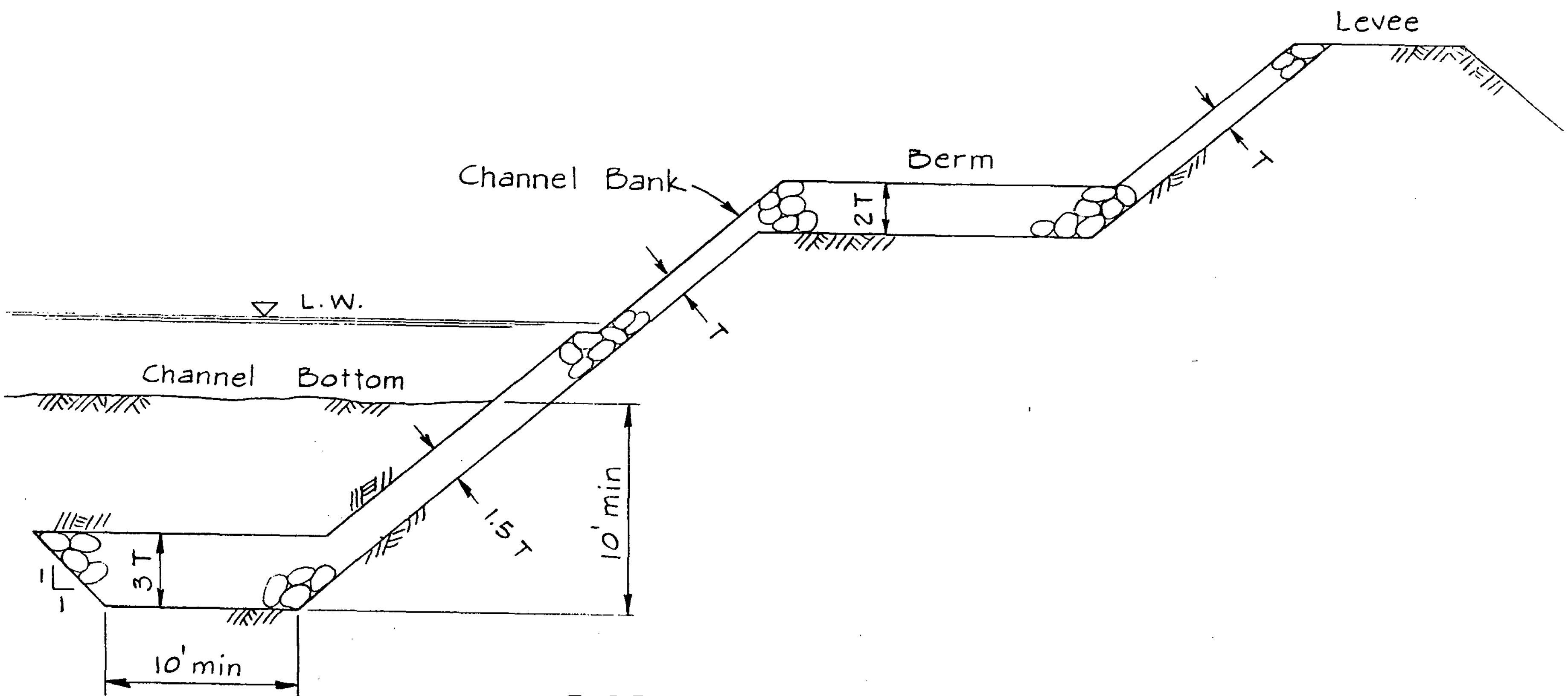
RIPRAP DETAIL

NOT TO SCALE

LOWER AMERICAN RIVER

LEVEE
ONLY

FIGURE 2



RIPRAP DETAIL

NOT TO SCALE

LOWER AMERICAN RIVER

CHANNEL BANK
& LEVEE

FIGURE 3

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 5

GEOLOGIC EVALUATION OF ALTERNATIVE DAMSITES

NORTH FORK AMERICAN RIVER

JANUARY 1989

PREPARED BY

GEOLOGY SECTION
GEOTECHNICAL BRANCH

GEOLOGIC EVALUATION OF ALTERNATIVE DAMSITES

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

TABLE OF CONTENTS (CONT.)

GEOLOGIC EVALUATION OF ALTERNATIVE DAMSITES - AUBURN DAM PROJECT

INTRODUCTION

Scope of Work

This section of the report will describe the results of a geologic evaluation of four damsites located on the North Fork American River near Auburn, California. The purpose of this study is to evaluate the pertinent geologic features associated with each site related to the possible construction of a dam at any one of those sites, and to provide data for cost estimate comparisons.

The scope of work included a literature search, data acquisition and analysis, geologic field reconnaissance, and preparation of this report.

Acknowledgments

The cooperation and assistance of the following agencies and individuals is gratefully acknowledged. Without their willingness to provide access to information in their files, this report could not have been prepared.

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Mr. Michael Schaefer

Mr. J. Wendel Carlson
Auburn, California
Former Auburn Dam Project Geologist (Retired)

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The Sacramento District Geology Section wishes to commend the USBR on the excellent and thorough geologic investigations that were accomplished on the Auburn Dam Project.

Summary Background of the Auburn Dam Project

Location. - This study discusses four damsites previously under consideration on the North Fork American River near the town of Auburn, Placer County, California. The sites are situated along the American River from the upper end of Folsom Reservoir, upstream to the confluence of the of the Middle and North Forks of the river at approximately river-mile (RM) 22.4, a distance of approximately 3.4 miles (see Figure 1). The sites are located at RM 19.0, 19.2, 20.1, and 22.1. The river-mile numbering system used when discussing the damsites throughout this report and earlier reports is from a U.S. Geological Survey (USGS) River Plan and Profile Survey of 1936.

The alternative sites were chosen by Bechtel National Inc. as part of a contract with the California Department of Water Resources for evaluation of the design and construction of facilities for the Auburn Dam Project. By examining topography and geology, Bechtel chose four sites based on two criteria, the suitability for construction of a dam large enough to impound 2.3 million acre feet of water, and a dam which would require less construction material than the proposed dam at RM 20.1 already examined by the USBR. Bechtel investigated a total of 12 alternative dam designs at the four sites. These are discussed in following portions of this report. It should be noted that Bechtel included consideration of a high dam at RM 22.1 only for cost comparison with similar size dams at the downstream sites.

The following paragraphs explain the sequence of events which led to the termination of construction by the USBR at the RM 20.1 site, and to the feasibility studies for alternative sites which followed.

Early Investigations. - Consideration had been given to constructing a dam along the lower portion of the North Fork American River as early as the 1920's. Perhaps the earliest record of detailed investigations is from 1929 when several prospective sites were examined by the American River Hydro-electric Company. Included were sites in the RM 17.9 vicinity, and at RM 21.0, 1.4 miles downstream of the confluence of the North and Middle Forks.

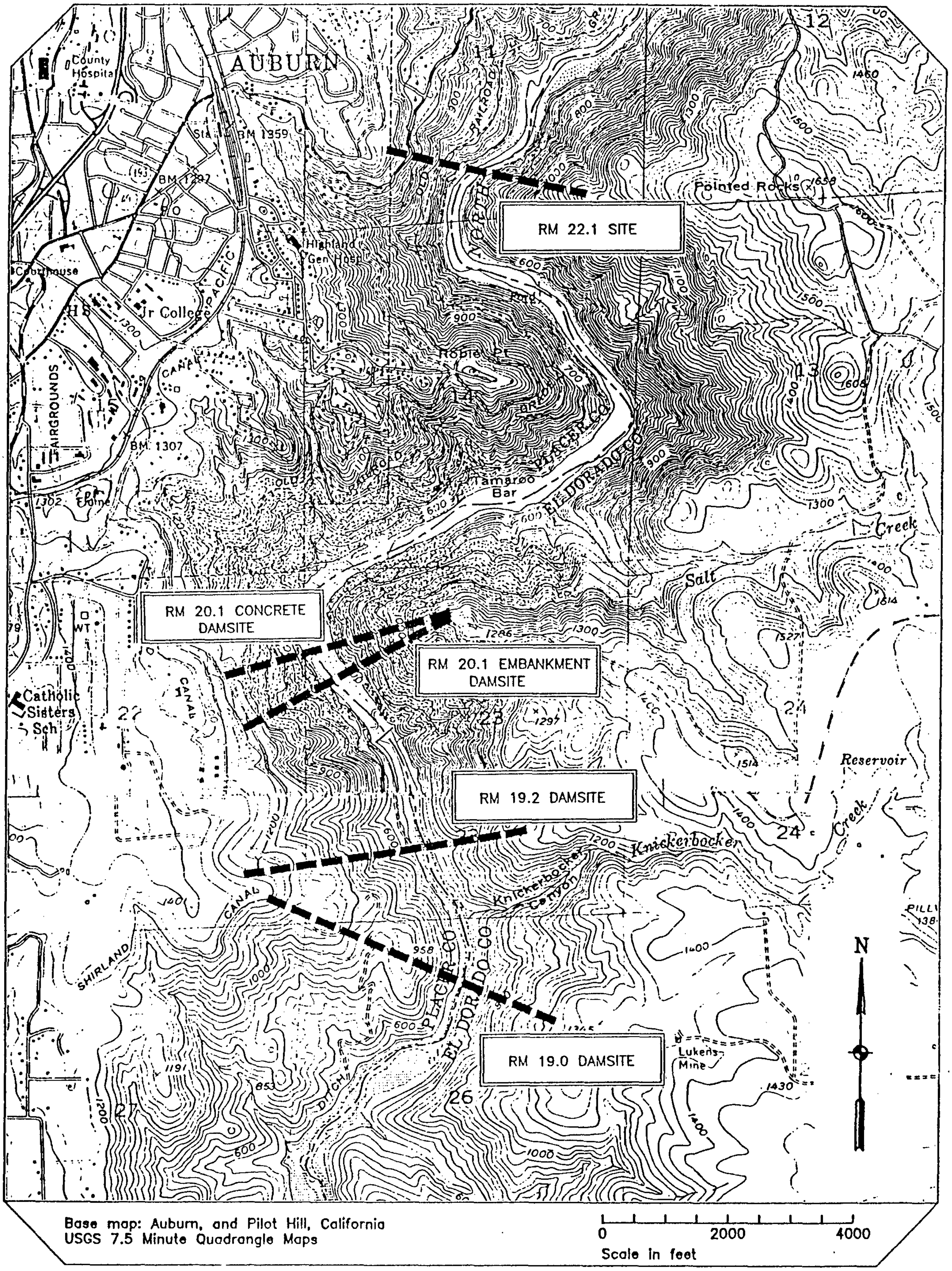

Figure 1
Location of Alternative Damsites

In June 1942, the USBR published a preliminary geologic report proposing construction of a concrete dam about 500 feet high at RM 22.4. The initial proposal for subsurface investigations included drilling 20 core holes, but an extensive landslide was encountered after completion of only seven of the holes, totaling 1,385 linear feet, and the site was eliminated from further consideration.

In 1943 the proposed site was shifted 4.5 miles downstream to approximately RM 17.9. That proposal consisted of a 520-foot-high dam with a 1-million-acre-foot reservoir. Six core holes were drilled at that site which was later flooded by Folsom Reservoir after completion of Folsom Dam in 1956.

A 1955 reconnaissance study conducted by the USBR concluded that a rockfill dam at RM 19.1, or a concrete dam at RM 20.5 were the most feasible alternatives to be considered. In 1956, the USBR drilled one core hole at Tamaroo Bar, at RM 20.5 (referred to as Robie Point in some reports), and concluded that a large zone of serpentine in the right abutment would provide poor foundation conditions for a concrete dam.

Subsequently, the emphasis was shifted to the RM 19.1 site. In 1957, the USBR drilled 30 holes, totaling 4,116 linear feet for the spillway, diversion tunnel, and outlet structures. The exploration program indicated that althought the rock contained numerous faults, shear zones, and other discontinuities, it was considered a suitable foundation for an earth or rockfill dam. However, they determined that the stuctural orientation of those discontinuities in relation to the steep canyon slopes produced conditions unsuitable for construction of a concrete dam. The results of those findings presenting the feasibility of constructing a dam at RM 19.1 were transmitted to Congress in early 1962.

In conjunction with the 1957 exploration program, the USBR conducted a construction materials investigation. It consisted of excavating 71 power auger holes and test pits, sampling cut-slopes and mine workings, and visual classification and laboratory testing of selected samples. The sources for impervious and semi-pervious borrow material for the dam embankment were to be from nine borrow areas, designated "100" through "900", all identified within a 3-mile-radius of the site.

In 1962 the design of the proposed dam was changed to that of a 690-foot-high structure with a reservoir capacity of 2,500,000 acre-feet, and its axis alignment was moved upstream approximately 1,000 feet to better fit the topography. A review of the geologic conditions and availability of construction materials indicated that the geologic explorations conducted during 1957 were adequate for the new site and no additional drilling would be necessary. Additional field mapping was conducted to delineate known and potential landslides and areas of instability, and to possibly extend the limits of the potential borrow areas to provide the additional embankment material needed for the enlarged dam. In late 1963, a supplemental report was forwarded to Congress discussing the USBR's conclusions regarding the proposed enlarged dam concept.

In 1965, following 2 years of Congressional hearings, construction of a dam was authorized at one of numerous sites between RM 17.9 and the confluence at RM 23.3.

Subsequent to that authorization, the USBR concentrated their efforts on a rockfill dam at RM 19.1 with an underground and/or surface powerplant, and a double-curvature thin-arch concrete dam at RM 20.1 with a surface powerplant.

In late 1966 and early 1967 the USBR conducted an additional materials investigation to better delineate the availability of dam embankment material within 10 miles of the damsite, and aggregate material within 20 miles. This study supplemented the 1957 materials investigation and included the possible use of gravel bar deposits on the American and Bear Rivers as aggregate sources.

Later in 1967, it was decided to design and build a double-curvature thin-arch dam at RM 20.1. That decision was based largely on the difficulty of obtaining sufficient material required for construction of a large embankment dam. Once the decision was made to build the thin-arch dam at RM 20.1 an extensive geologic investigation and testing program was undertaken. The following paragraphs discuss that investigation program.

River Mile 20.1 Site. - The Auburn field office of the USBR was established in 1966, prior to the final decision on the design of the dam.

Once the final design for the configuration of the concrete dam was established it was decided that the foundation mapping and detailed geologic investigations would include the area 500 feet upstream and downstream of the axis of the dam, and to a depth of 500 feet below the dam foundation.

Site geologic investigations began with detailed mapping of surface exposures. Because most of the surface was obscured by residual soil and slopewash, approximately 5 miles of exploratory dozer trenches were excavated to expose the near-surface geologic features. During the same time period, extensive NX core drilling was being conducted to explore the subsurface features.

As part of the overall geologic investigation program, a series of exploratory tunnels, drifts, and raises were excavated into the abutments at various elevations beginning in December 1967. The design of the underground excavations was based largely on the complex nature of the geologic structure exposed at the surface. It consisted of six main tunnels with a total length of 3,550 feet, eleven drifts within the tunnels totaling 2230 linear feet, five raises totaling 1028 linear feet, and a 10.5-foot-diameter shaft 185 feet deep. For testing purposes, the drifts were excavated at right angles to the anticipated principle stresses to be exerted by the dam.

Analysis of geologic information from the exploration program delineated three major structural units; continuous fault zones

(F-zones), talc zones (T-zones), and blocks of foundation rock bounded by the fault and talc zones.

The underground explorations were then used to conduct in-situ rock mechanics and laboratory testing to determine deformation characteristics within various foundation blocks as related to variation in rock type and the effects of jointing and minor shears. Testing was also conducted to determine deformation and shear properties of the fault and talc zones. The testing program for the rock consisted of 18 uniaxial and three radial jacking tests on the blocks. At most sites, two surfaces were prepared, giving a total of 33 values. The portion of the program conducted on the discontinuities consisted of 22 in-situ plate gouge jacking tests to determine deformation properties of the F-zones and T-zones, and six direct shear tests to determine shear strength, cohesion, and friction values of the discontinuities. The results of the testing is discussed in Appendix A, <u>Foundation Properties</u>.

In conjunction with the in situ testing of the foundation rock, extensive laboratory testing of rock core samples and undisturbed shear zone samples obtained from the in situ testing sites were conducted.

By 1972, approximately 83,000 linear feet of drill core had been obtained from 304 holes drilled from both the surface and the tunnels for the foundation investigations alone.

Permeability testing was performed to determine the permeability of the bulk rock as well as the F-zones and T-zones. High-pressure exit gradient tests were conducted on the discontinuities within the tunnels to determine their piping potential beneath the dam. In situ testing within the underground excavations was completed in November 1969.

During the excavation and cleaning of the foundation keyway extensive geologic mapping was conducted.

The change in the design from that of an embankment dam to a concrete dam required additional investigations to identify aggregate sources for the approximately 6 million cubic yards of concrete required for the thin-arch concept. Late in 1967 and again from 1968 through 1970, the USBR conducted investigations to identify the quantity and quality of the aggregates in the gravel deposits located within the proposed reservoir area. The studies estimated that approximately 8 million cubic yards of pitrun aggregate could be obtained from approximately 270 acres of gravel bars and river channel.

<u>Seismic Studies</u>. - On August 1, 1975 a Richter magnitude 5.7 earthquake occurred near Oroville, California, about 45 miles north-northwest of Auburn. That earthquake produced surface cracking on what is considered to be a northern extension of the Bear Mountains fault zone, portions of which also pass near the Auburn damsite. As the result of that earthquake, numerous seismic safety studies were undertaken to determine, 1) the seismic potential of the area with

emphasis on estimating the Maximum Credible Earthquake (MCE) for the site, 2) the ground motion that would be produced by this MCE, 3) whether reservoir-induced seismicity is possible, and 4) if there are faults in the dam foundation along which displacements could occur in the event of an earthquake. The following summaries describe the various major reports from the seismic studies with their results and conclusions.

Note: The numbers in parenthesis ( ) throughout this report refer to the corresponding entries in the Bibliography (Section IV).

1. Bureau of Reclamataion Project Geology Report (13). In September 1975, shortly after the Oroville earthquake, the USBR initiated a seismic evaluation of the Auburn area to determine the potential for surface faulting in the foundation of the dam. This was the preliminary study focused on the structural and stratigraphic relationships of T-zones, F-zones, and intrusive dikes in the site area. The study included approximately 20 miles of dozer and backhoe trenching, and 47 core drill holes totaling about 20,000 linear feet in the area surrounding the damsite. Fault zone (F-1), the largest and longest (4,500 feet) fault present in the foundation for the arch dam, was the focus of this study. F-1, which due to its relationship with the T-zones, could be used to determine the relative ages of the discontinuities. The entire length of F-1 was exposed in and outside the foundation to determine the age of last movement.

The results of the studies were published in 1977 in a three volume report entitled, "Seismic Evaluation of Auburn Damsite" (13). The major conclusions of this study were:

1. F-zones, the designation for the system of local faults which cross-cut the regional structure, offset all T-zones which parallel the regional structure. Hence, F-zones are the youngest discontinuities in the foundation and appear to be unrelated to the regional metamorphic structure. Based on these relationships, the USBR investigators (as well as the independent consultants reviewing the seismic investigations) concluded that displacements along T-zones in the foundation are not credible.

2. Studies of F-1 show that although as much as 120 feet of lateral separation of early Mesozoic age rock is documented in the foundation, no more than a few feet of displacement has occurred since dike emplacement, 130 to 140 million years ago. Later investigation provided more finite limits on F-1 movement.

3. A fault discovered near the ridge top southwest of the dam foundation early in the investigations of regional faulting, was later shown to displace a geologically young (5 to 20 million years old) volcanic deposit capping the metamorphic bedrock in the area. This fault, termed the Maidu East shear zone, was subsequently shown to be inactive by USBR standards. This assessment was based on the presence of buried soil layers, or

paleosols, judged to be at least 100,000 years old. These soils are traceable across the Maidu East shear zone and show no evidence of displacement by the fault.

2. Woodward-Clyde Consultants (15). In March 1976 Woodward-Clyde Consultants (WCC) made an unsolicited proposal to USBR to study the age of faulting in the Auburn area. WCC was awarded a contract to conduct an independent evaluation of the potential seismicity of the Auburn damsite. Their work was completed in July 1977 with the submittal of the eight volume report titled "Earthquake Evaluation Studies of the Auburn Dam Area". The basic results of their study was, 1) that an estimated MCE of Richter magnitude 6 to 6.5 at a focal depth of 6 miles could be expected "near" the damsite, 2) that numerous structures in the area have the potential for dip-slip surface rupture with a maximum estimated displacement of 0.8-foot, and 3) that there would be a 50 to 80 percent likelihood of a reservoir induced seismic event (RIS) greater than magnitude 3.0 and a 30 percent chance of a RIS of the magnitude of the Oroville earthquake (5.7) or larger occurring near the dam during the design life of the dam.

3. Bureau of Reclamation Panel of Consultants (19,20,21,22,23). A panel of five consultants was retained by the USBR in April 1976 to review and evaluate the seismic studies conducted by the USBR (13), the WCC report (15), and the static and dynamic design and analysis of the dam (16). This panel of nationally recognized experts in the fields of geology, seismology, and dam design were Dr. Richard Jahns, Dr. R.W. Clough, Dr. Clarence Allen, Dr. Lane Johnson, and Dr. J. Laginha Serafim. At the completion of the seismic studies in July 1978, each consultant presented a brief report detailing their conclusions and recommendations. These reports were reproduced and distributed as a part of the USBR's series of reports on the Auburn Dam seismic studies.

4. Supplement to Project Geology Report by the Bureau of Reclamation (17). During the height of the USBR's Auburn vicinity seismic investigation, it became clear that several areas of highly specialized studies requiring outside expertise were warranted. Because the completion of these studies extended beyond the original time estimates for the site investigations, a two-stage program of data release was selected. The results of the first stage of investigations was presented in the 3-volume Project Geology Report (13) published in mid-1977 (discussed above) while the results of the second stage studies were produced in a 6-volume series published in mid-1978 titled "Seismic Evaluation of the Auburn Damsite" (17). Following is a brief discussion of the more critical reports:

Volume 1, "Evaluation of Quaternary Stratigraphic Data for Assessing Fault Activity, Maidu East Shear Zone", by consultant Dr. Roy J. Shlemon. This report evaluates the conflicting WCC and USBR evidence of fault activity on the Maidu East shear zone. Dr. Shlemon concluded that 1) the local bedrock steps used by WCC to corroborate evidence of faulting are the result of soil development, not faulting, 2) the buried soil in trench GT-1 which

WCC used to demonstrate active faulting is actually a buried soil tongue, not a paleosol, and is not displaced across the Maidu East shear zone, and 3) the two 100,000-year-old buried paleosols found in trench DT-65 (after WCC completed their field work) were unbroken across the Maidu East shear zone and demonstrate that this fault "..has been inactive for at least the last 100,000 years".

Volume 3, "Study of Dike/Fault Intersections Northwest Portion of the Auburn Damsite", by consultants D.K. McMillan and J.D. O'Brient. This report presents detailed analyses of the structural relationships between faults F-1 and F-0 and the cross-cutting, age-dated dikes exposed northwest of the damsite. This study provided the precise direction of last movement occurring on F-1 which was needed to determine the amount of post-quartz mineralization displacement on F-1 in Volume 4.

Volume 4, "Analysis of Faulting in the Auburn Damsite" by staff geologists D. Ostenaa and R.H. Throner. This report documents the various analytical steps utilized in determining the amount and timing of the last known movement of fault F-1. The key element in this study is a left abutment foundation shear, identified as the Steeply Dipping Shear (SDS). This shear parallels bedrock foliation and is the only shear which offsets fault F-1. This relationship and SDS's structural relationship with other intersecting quartz veins, shears, and dikes permitted investigators to demonstrate through the use of orthographic solutions that only about 2.5 feet of net slip displacement has occurred on F-1 since the period of quartz mineralization, about 100 to 120 million years ago. This key assessment provided the geotechnical members of the Auburn consultants panel the means to determine meaningful amounts of potential fault displacements and earthquake recurrence intervals for the Auburn site.

5. Review of WCC Report by USGS (49). The U.S. Geological Survey reviewed the WCC Report in 1978 and concurred with the findings that: activity on the foundation faults were indeterminate by USBR criteria; the Maidu East fault exhibits evidence of displacement within the last 100,000 years (i.e. active by USBR criteria); that there was likelihood of reservoir-induced seismicity; and that an epicenter within 0.5-mile of the dam should be considered. However, the USGS concluded that a Magnitude 6.5 to 7 earthquake with a 3-foot displacement in the foundation was possible rather than a Magnitude 6 to 6.5 earthquake with 0.8-foot displacement recommended by WCC.

6. Review of Seismic Safety of the Auburn Damsite by CDMG (50). This 1979 report, which was issued as the California Division of Mines and Geology Special Publication 54, was prepared as a joint effort by California's Division of Safety of Dams, Department of Water Resources, and the Consulting Board for Earthquake Analysis of Auburn Dam. The report served as the State's official position on the geologic and seismic parameters to be used in the dam design. The first portion of this publication is a report by the Consulting Board for Earthquake Analysis for Auburn Dam, a

consulting board of eminent geologists, seismologists, and dam design engineers engaged by California Department of Water Resources, This board, chaired by George W. Housner with members John H. Blum, Bruce A. Bolt, Douglas D. Campbell, Alan L. O'Neill and H. Bolton Seed, concluded that the following design parameters for a dam at the Auburn site were appropriate:

1. A Magnitude 6.5 earthquake with a response acceleration of 0.50g in the one second portion of the spectrum.

2. A fault slip in the foundation rock of up to 5 inches.

The State Geologist, together with the CDMG staff, concurred with the Board's design parameter except in the area of foundation displacement. CDMG concluded that 9 inches of foundation displacement is a reasonable design parameter.

7. Department of the Interior Press Releases.

July 30, 1979: In this press release Secretary of the Interior, Cecil Andrus announced that the seismic design parameters for Auburn Dam would be those recommended by the State of California. These were: 1) and MCE of Richter Magnitude 6.5; 2) a foundation displacement of up to 9 inches in a single seismic event; and 3) a ground response acceleration of 0.50g in the one second portion of the spectrum.

December 30, 1980: In this press release, Secretary of the Interior, Cecil Andrus announced that a "safe dam can be constructed at the Auburn site". He also recommended that a concrete gravity dam be selected rather than a rock-fill embankment dam.

8. Additional Reports. Several other reports were written to cover various design aspects of the dam which are beyond the scope of this study. A partial list of those include:

Design Analysis of Auburn Dam (16)

Paleomagnetic Investigation of F-1 Fault, Auburn Dam (30)

Auburn Damsite Seismic Studies Overview (18)

Auburn Damsite Seismic Studies Summary (29)

USBR Proposed Alternatives. - In January 1979, following the seismic studies, the Secretary of the Interior halted construction on the thin-arch dam, and the USBR undertook studies of alternative designs at RM 19.1 and 20.1. The studies considered embankment and concrete dams which would provide the same reservoir storage and power generating capabilities as the thin-arch dam. Based mainly on the existing work that had been done at the RM 20.1 site, the

designs were narrowed to two alternates; a double-curvature concrete gravity arch dam (CG-3) at mile 20.1, and a rockfill dam at RM 20.1 just downstream of the existing thin-arch keyway at RM 20.1

Concrete Curved-Gravity Dam.- The foundation for CG-3 was designed to utilize the existing foundation excavation for the thin-arch dam with some additional excavation required downstream of the keyway. The foundation for the alternate design, like that of the thin-arch dam, contained structural discontinuities in the form of joint sets, cleavage planes, shear zones, and fault zones, including the major fault zone (F-1) which traverses most of the left abutment foundation.
Treatment of the discontinuities, deformable rock, and differentially weathered zones conducted on the thin-arch foundation would have been extended downstream into the CG-3 foundation excavation as necessary.

The CG-3 design would have been 685 feet high, with a crest length of 4,150 feet, and a base thickness of 465 feet. It would have had a gated spillway located in the center of the dam.

Rockfill Dam. - The designs for the rockfill alternative included a central core rockfill embankment with a excavated surface spillway, river outlet works, and an underground powerplant. The foundation would have been located approximately 1,400 feet downstream of the axis of the RM 20.1 thin-arch axis. The USBR determined that the foundation for the dam and spillway was comprised of the same type of rock as the thin-arch foundation, but was geologically less complex with fewer T-zones and F-zones.

Because the rockfill dam would utilize much of the excavations conducted for the thin-arch dam, detailed mapping of geologic structures and knowledge of foundation conditions were already available. Additional geologic investigations included drilling for a new powerplant and spillway alignment, and foundation explorations on the upper right abutment.

Bechtel Proposed Alternatives. - In February 1984, a Federal-State task force was organized for the purpose of reviewing the status of the Auburn Dam Project and its viability in the region's water and power development. This task force (The Auburn Dam Task Force) sought outside assistance to determine two issues. First, was the project as proposed by the USBR the least costly to accomplish the desired functions? Secondly, is there a smaller-sized project acceptable to non-Federal investors? To this end, the California Department of Water Resources (DWR) retained Bechtel to prepare an evaluation of the Auburn Dam Project. The funding of this project was shared equally by DWR and USBR. Following is a summary of the results of the study titled, "Evaluation of the Auburn Dam Project" completed in November 1985.

Bechtel considered four different types of dams; concrete gravity similar to the USBR's CG-3 design, roller compacted concrete (RCC), rockfill, and a concrete-faced rockfill dam. A total of 12 alternatives to the CG-3 dam were evaluated for sites at RM's 22.1,

20.1, 19.2, and 19.0. The concrete dams would have had spillways located on the main river portion of the dam. For the rockfill alternatives, a stepped spillway would be constructed on one of the abutments.

River Mile 22.1. - The steep canyon walls at RM 22.1 would allow for a dam with a relatively short axis. Bechtel evaluated a concrete gravity dam which would require approximately 6.2 million cubic yards of concrete, an RCC dam with a volume of 10.3 million cubic yards, and a rockfill dam with a central core which would require about 34.6 million cubic yards of material, all of which would have a straight axis from 2750 to 2770 feet long.

Bechtel noted that the disadvantages to this site are; the need for diversion tunnels, the need for a conduit from the dam to the Placer County Water Agency's (PCWA) intake tunnel, and the fact that a large dam would flood-out the PCWA's Oxbow Powerplant on the Middle Fork American River at times of high reservoir levels.

Bechtel considered the landslide on the right abutment, which was discovered by USBR during their studies for the RM 22.4 site in 1942, and concluded that by moving the axis alignment to RM 22.1 a conventional concrete and an RCC dam could be built without encountering the slide.

Bechtel was of the opinion that if a rockfill dam was built at this site the material in the slide could be removed and used as embankment material.

River Mile 20.1. - At RM 20.1, Bechtel evaluated all four alternatives. They considered alternatives which would take advantage of work already accomplished at the site, ones in which the axes would be shorter than the CG-3 dam, and if possible, ones that would avoid the F-1 fault.

The first design considered was that of a concrete gravity concept aligned to avoid the landslide area on the right abutment and to not be located on the F-1 fault, except on the extreme left end of the dam where the height would be only about 30 feet. The length of the dam would be 3450 feet and would contain approximately 8.3 million cubic yards of concrete. No modification to the existing diversion tunnel would be required.

An RCC dam, also considered by Bechtel, would also have a length of 3450 feet but due to the thicker section would have a total volume of approximately 13.6 million cubic yards. Although some foundation excavation and treatment would be required, the diversion tunnel could be used without modification.

Bechtel also considered a rockfill dam with an impervious core slightly downstream from the axis alignment selected for the concrete gravity dam. The dam would be 3380 feet long and have a total volume of approximately 42 million cubic yards. Their reason for moving the axis alignment downstream was to keep the upstream toe inside the existing cofferdam. Although the cofferdam was

partially destroyed during the 1986 flooding and would require rebuilding, the location of the cofferdam would not change.

Because of the wide footprint of the foundation, the rockfill dam would cover the area occupied by the slide on the right abutment and several hundred feet of the F-1 fault. The slide material which would require removal may be usable as embankment material.

The downstream toe of the dam would extend beyond the outlet of the existing diversion tunnel and would require the construction of a 1400-foot-long tunnel to extend it.

The last type of dam considered was a rockfill dam with an upstream concrete face. The axis alignment for this dam would also be shifted downstream to keep the face of the dam away from the F-1 fault, but by doing so would result in the dam being located on the right abutment landslide. This would also require removal of the slide material. The foundation would be 3380 feet long and have a volume of about 37 million cubic yards.

The concrete-faced rockfill alternative would also require construction of a 1200-foot-long extension to the existing diversion tunnel.

The concrete-faced rockfill alternative would require excavation of a spillway through the ridge forming the left abutment. The spillway would be an unlined, stepped chute approximately 1400 feet in length.

Bechtel determined that there was no cost savings to construct a rockfill dam with a concrete face instead of a conventional rockfill dam, so the concrete face concept was not considered at any other location.

River Mile 19.2. - Bechtel considered a concrete gravity and a rockfill dam at this site. The entire construction area would be located within the limits of Folsom Reservoir and would thus require construction of a downstream cofferdam. They also reported that either the existing diversion tunnel would need to be lengthened and routed past the downstream site by way of a conduit, or that an upstream cofferdam and a diversion tunnel would need to be built.

Because of the relatively steep canyon walls, a dam at this site would need to be approximately 3380 feet long to impound 2.3 million acre feet of water. The volume of materials required for dams at this location would be approximately 6.5 million cubic yards for a concrete gravity dam and approximately 35 million cubic yards for a rockfill dam. It is not known why Bechtel didn't consider an RCC dam at this site.

The spillway for a full-sized rockfill dam at this site would be located on the right abutment. It would consist of a gated, unlined stepped chute approximately 1200 feet long. The maximum depth of excavation would be approximately 120 feet at the spillway crest.

An item of concern which Bechtel didn't address in their studies was the need for the care and diversion of water exiting Knickerbocker Canyon. Rockfill dams at both RM 19.2 and 19.0 would require some type of diversion structure to carry water away from the embankment which would be constructed across the mouth of the canyon.

River Mile 19.0. - The types of dams considered at this site were a concrete gravity, an RCC gravity, and a rockfill. Due to the configuration along this reach of river, the right abutments would be located very close to those of the RM 19.2 site. The length of the dam would range from 3900 feet for the rockfill dam to 4030 feet for the concrete dams. Like the RM 19.2 site, any of the alternatives at this site would require a downstream cofferdam to impede the backwater from Folsom Lake.

Bechtel selected an RCC dam at this site as being the most cost effective full-sized alternative to the CG-3 dam at RM 20.1, Their determination was based mainly on the slightly narrower canyon at this site and the resultant reduction in volume of construction materials needed.

The USBR evaluated the Bechtel report and noted in several memos that there were items in the report which were misleading when not clarified. The most important of those was the fact that although Bechtel included the cost for site investigation in their determination of the most cost-effective alternative site, they didn't include the time needed for that investigation. USBR Geologist Wendel Carlson (35) pointed out that:

> "...the RCC type dam at mile 20.1 is $91.4 million more than the RCC type at 19.0, but can be completed at an estimated 34 months earlier. This 15 percent increase in cost, although significant, does not appear excessive if the mile 20.1 dam can be completed nearly three years ahead of the mile 19.0 dam. A similar comparison of the rockfill designs shows that at mile 20.1 site, the cost is only about 2 percent greater, but the completion time is 26 months earlier."

Note: The 34 months difference in the completion time is derived from RM 19.0 requiring an additional 17 months of site preparation, and 24 months of geologic investigations, but with 7 months less actual construction time for the dam, spillway, and power facilities.

GEOLOGIC STUDY

Purpose

The purpose of this study is to present findings of geologic concern for the proposed alternative damsites (RM 22.1, 19.2, and 19.0 sites) to the USBR 20.1 Auburn Damsite for design and estimating construction costs. The RM 20.1 damsite is not addressed in detail in this report because the geology and seismicity of the site have been studied extensively, and there are no known site conditions which would significantly affect the dam design and estimation of construction costs.

The geologic review of the proposed damsites was accomplished in three steps: 1) preparation of a bibliography; 2) collection of available pertinent data; 3) analysis of geologic data for each damsite. Those studies culminated in the preparation of this report.

Bibliography. - In preparation of the bibliography, all available geotechnical, environmental, and design data written for the Auburn Dam Project was researched. This included reviewing correspondence and technical reports prepared by five agencies in the Sacramento area to locate and identify the data. Those agencies included: 1) U.S. Bureau of Reclamation; 2) California Division of Mines and Geology; 3) California Department of Water Resources; 4) California Department of Water Resources Division of Safety of Dams; and 5) U.S. Army Corps of Engineers.

The references cited in this report are on file in the Geology Section of the Geotechnical Branch and are included in this report as Chapter IV, Bibliography. A complete reference relating to the Auburn Dam Project from 1929 to 1987, along with the location of each reference in the Sacramento area, follows the report as Appendix B, Auburn Dam - Geotechnical, Environmental, and Design References.

Data Acquisition. - Available pertinent geotechnical and design data on the alternative damsites were acquired for review from the U.S. Bureau of Reclamation Mid-Pacific Region Office and the Auburn Construction Office.

Geologic Considerations. - The geologic analysis used in review of the proposed alternative damsites included consideration of: 1) spillway alignments; 2) tunnel alignments; 3) landslides; 4) dam foundation alignments; 5) faults and seismicity; 6) site access and clearing; 7) borrow areas; 8) environmental impact; 9) geologic investigations; and 10) scheduling and staging. The following paragraphs describe the details of geologic considerations that were addressed in the analyses of the alternative damsites.

Spillway Alignments. - Spillway alignments at the damsites are proposed only for embankment dams since conventional gravity and roller compacted concrete dams have the spillways incorporated into the structure. The geologic analysis of the spillway alignments includes the determination of the lithology, depth of weathering, and nature of discontinuities necessary for determining design criteria, quantities of excavations, design of cut-slopes and the overall associated costs. For example, concrete lined spillways will be required unless the invert is excavated into slightly weathered or fresh rock.

It cannot be assumed that all material excavated from the spillway will be suitable for use in an embankment dam. Specific rock strength, density, and gradation are required in a structure of this type. In cases where rock is highly weathered and soft, the excavated material may be suitable for use in an impervious core with special processing and treatment.

Tunnel Alignments. - Diversion tunnels are not necessarily required during construction of a concrete gravity dam. An alternative is a staged construction in conjunction with a river outlet-works where the diversion feature is incorporated into the dam and later plugged. In another case, a single tunnel can serve as an interim diversion facility and later as a river outlet facility.

Although diversion tunnels for embankment dams are ideally located along alignments which require a minimum of excavation, consideration must be given to lithology, weathering, discontinuities, and ground water. Longer alignments in hard unfractured rock may be less expensive than shorter alignments in rock which is highly weathered, highly fractured, and crushed or sheared and therefore require specialized tunneling techniques. Special consideration must also be given to tunnels which cross faults that may have potential for displacement.

Landslides. - Landslides present difficulties for dam construction and operation. During construction, landslides in the dam foundation must be removed. It cannot be assumed that the landslide debris will be suitable as a construction material. For embankment and RCC dams, special processing and treatment of the debris may be required. Specific rock strength, density, and gradation are required in a design of this type. For conventional concrete dams, all excavated debris is wasted. Landslides located adjacent to portals of river outlet works or diversion tunnels have a potential for further movement or complete failure (especially during reservoir operation) resulting in blockage of tunnel portals. When possible, tunnel portals are located in areas not affected by landslides. If unavoidable the landslide or slopes having potential for failure are removed.

Dam Foundation Alignments. - Dam foundation alignments are important in determining estimates for excavation and associated costs. Concrete dams (conventional and roller compacted concrete) require excavation to the top of slightly weathered rock.

Embankment dams constructed on rock only require excavation to the top of moderately weathered rock.

For concrete dams all excavated material is wasted. For rockfill dams it cannot be assumed that the excavated foundation materials will be suitable for use as a construction material in the dam. Specific rock strength, density, and gradation are required in a design of this type. However, in the extreme case where the rock is highly weathered and soft, the excavated material may be suitable for use in an impervious core. Special processing and treatment may be required.

Special consideration is given to discontinuities in the rock of the excavated foundation. These discontinuities include fractures, joints, shear zones and faults. In all dam designs (concrete and embankment dams) overexcavation of soft and crushed zones in shears and faults is required. Faults are discussed in more detail in the following section. In addition, grouting requirements (grout curtain design) are dependent on the depth, spacing, and tightness of fractures and joints. All grout curtains and foundations are designed to minimize the velocity and increase the flow path of water moving through the foundation.

Faults and Seismicity. - Regional and site-specific faults and seismicity are important in selecting the location of damsites. Seismicity and the structural relationship of faults are evaluated to determine the expected maximum intensity of ground-motions and fault displacement in the dam foundation.

A dam foundation and structure are designed to withstand seismic loading such as ground-motions and fault displacement produced by a MCE. An MCE is the largest earthquake that appears capable of occurring under the given geologic conditions. It is a rational and believable event that is in accord with the present knowledge. MCE's are commonly assigned to various sources. Large magnitude earthquakes from distant sources will not necessarily produce ground motions as intense at a given damsite as a smaller magnitude earthquake occurring at the damsite. Generally, the potential for ground rupture and fault displacement is greater from a site-specific earthquake.

All faults in the dam foundation are not necessarily considered as being capable of activity or displacement. The criteria for determining fault activity are based on historical seismicity, structural relationship to known active faults, and the last dated movement and recurrence of movement along a fault. Different agencies have different parameters for defining the last dated movement and recurrence of movement along a fault. Faults determined to be active by the U.S. Army Corps of Engineers define the last dated movement as being less than 35,000 years old. Faults determined to be active by the USBR are ones determined to have experienced relative displacement during the last 100,000 years.

Reservoir induced seismicity is a relatively new concern for deep reservoirs. Reservoir induced seismicity is seismicity which may occur within the reservoir as a result of loading and or lubrication of faults by water as reservoir impoundment occurs. WCC reviewed geologic and seismologic data for 16 dams and reservoirs in the Sierran foothills. Of those 16, six are deep and/or large reservoirs in geologic and structural settings similar to the proposed Auburn Dam and reservoir. Only one of the six, Oroville, is suspected of having reservoir induced seismicity and this is considered "questionable" (15). However, it is generally considered that any seismicity induced by reservoir impoundment will not exceed that which can be produced by a site-specific MCE.

__Site Access and Clearing__. - Site access and clearing are important in terms of property ownership and physical features. Investigations beyond feasibility level will require extensive road development leading into the site and at the site. Once a site is selected for construction, clearing of vegetation and stripping of overburden in the foundation is required. An embankment dam requires stripping an area much larger than that required for construction of either a conventional or RCC gravity dam.

__Borrow Areas__. - Sources of construction material are important for concrete, and embankment dams. Ideally, the sources of borrow are available in the proximity of the damsite. For concrete dams nonreactive hard aggregate is required. For embankment dams a rock source and an impervious material source are required (except for a concretefaced rockfill dam). The borrow sources ideally should be located in areas which have a minimum affect on the environment. Locating borrow areas in the reservoir area would have the minimum environmental impact.

__Environmental Impact__. - The affect on the environment in selecting a damsite involves broad areas of concern. The most visible are concerns with borrow areas, waste areas, and reservoir rim slope stability. Other areas of concern involve fish, wildlife, vegetation, water quality, and water rights (both ground water and stream flows).

__Geologic Investigations__. - Geologic investigations are crucial in dam design and foundation design and are generally performed in conjunction with a regional fault and seismicity study. The site investigations would generally include reconnaissance field mapping, several exploration core holes, and limited trenching. Once a site is selected, detailed mapping, extensive core drilling, extensive trenching, and in-situ rock mechanics testing is performed in conjunction with a detailed site-specific fault and seismicity study. The RM 22.1, 19.2, 19.0 sites would have to be explored based upon the concept of a large dam in order to determine whether an expandable structure could be built there. The time required to perform these explorations would be a minimum of 2 years and depending upon results of the exploration and evaluation, could be longer.

Table M-5-1 shows a comparison of the status of geologic investigations conducted for the full-sized alternatives. The table was prepared by USBR (35) upon reviewing the Bechtel report (47).

__Scheduling and Staging__. - Staging dam construction for future expansion is a relatively new concept. However, older existing dams have been modified and raised to increase reservoir storage for the purpose of flood control or water use. Spillways on existing dams have also been modified for the purpose of flood control.

Building a dam for the purpose of future expansion requires the same level of geotechnical investigation and design as that which would be required for the final long-term structure. This includes foundation, tunnels, and spillway concerns. In addition, sources of borrow (both aggregate for concrete dams, and rock and impervious material for embankment dams) are evaluated. If borrow sources are located in the reservoir, the materials used in future expansion may require stockpiling at the reservoir rim or at some downstream location. An exception to this would be the case of expanding a "dry dam" design to impound water.

Regional Geology, Faults, and Seismicity

The following paragraphs briefly describe the regional geology, faults, and seismicity in the vicinity of the alternative damsites. An in depth discussion of these topics is available from sources listed in the references section of this report.

__Regional Geology__. - The proposed Auburn Dam sites are located in the western foothills of the central Sierra Nevada Geomorphic/Geologic Province. The Sierra Nevada is a highly asymmetric mountain range having a long gentle western slope and a high steep eastern escarpment. It ranges from 50 to 80 miles wide, is about 400 miles long, and trends northwesterly. The alternative damsites lie within a portion of the foothills referred to as the "Western Metamorphic Belt". This belt is a northwest-trending zone 30 to 50 miles wide and 250 miles long. The eastern margin of the belt is delineated by intrusive rocks of the Sierra Nevada batholith. The western margin of the belt is overlain by sedimentary rocks of the Great Valley sequence.

The geologic history of rocks in the Western Metamorphic Belt is long and complex and beyond the scope of this report. An accepted explanation involves plate tectonic concepts. During the Mesozoic Era the western margin of the Sierra Nevada underwent several periods of intense crustal deformation. The North American plate converged with the Pacific plate resulting in the accretion of terranes comprising the Western Metamorphic Belt and in the formation of the Foothills fault system.

TABLE M-5-1

STATUS OF GEOLOGIC INVESTIGATIONS

Site	Type of Dam	Status of Mapping	Exploration Trenches Logged (Feet)	Site Drilling No. Holes	Site Drilling Footage	Exploration & Test Tunnels (Feet)	Rocks Mechanics Testing
20.1	CG-3 (Curved axis)	All construction excavation geologically mapped in detail	26,000±	372	98,000±	6,958	Completed
	CG or RCC (Straight axis)	Geologic mapping, exploratory trenching, drilling, tunneling, and rock mechanics testing at CG-3 is applicable at this site.					
	Rockfill (Concrete face or conventional)	As above, but some additional drilling would be required.					
19.0	CG or RCC	Left abutment- None	None	None	-0-	None	None
		River channel- Incomplete	None	None	-0-	None	None
		Right abutment- Incomplete	8,500± *	19 *	4,600±	None	None
	Rockfill	As above	15,000±	30 (right & left abutments)	8,000±	None	None
19.2	CG	Preconstruction grade geologic mapping nearly complete	9,300±	23	6,200±	None	None
	Rockfill	As above	15,000±	30	8,000±	None	None
22.1	CG, RCC, or Rockfill	Incomplete	None	7	1,400±	None	None

* Majority of holes located outside the immediate area of the proposed axis for the dam.

The Western Metamorphic Belt consists primarily of Paleozoic and Mesozoic metamorphic marine sedimentary and volcanic rocks ranging in age from 150 to more than 300 million years old. The rocks are complexly folded and faulted. Bedding and foliation are mostly steeply dipping to vertical and trend parallel to the Belt. The sequence is locally intruded by Mesozoic granitic rocks which comprise minor rock types. Tertiary volcanic and sedimentary materials ranging in age from 5 to 40 million years fill paleo-drainages. Remnants of Plio-Pleistocene formations ranging in age from 1 to 5 million years blanket portions of the Belt. Present-day drainages dissect rocks of the Western Metamorphic Belt.

Faults. - The Foothills fault system consists of northwest-trending, subparallel, near-vertical fault zones. The faults are located within the Western Metamorphic Belt, and divide it into several large terranes (blocks). The fault system includes two major fault zones in which ultramafic rocks are closely associated. Rocks along the zones have been locally altered to serpentine, talcose serpentine, talc schist, and chlorite schist. South of Placerville, the fault zones are generally well defined linear features having relatively few structural complexities. North of Placerville the fault zones branch out, forming a network of structurally complex and less well-defined systems. The easternmost fault zone is referred to as the Melones fault zone. It projects southeastward to approximately 9 miles east of the damsites. The westernmost fault zone is referred to as the Bear Mountains fault zone. It branches and projects through the vicinity of the damsites.

The last major movement along the Foothills fault system occurred in response to the tectonic regime in existence during the Mesozoic Era about 140 million years ago. Other significant movement along the fault system occurred approximately 65 million years ago. Some faults within the Foothills fault system have been reactivated since late Cenozoic time, beginning approximately 5 to 10 million years ago.

Branches of the Bear Mountains fault zone are not well defined in the vicinity of the damsites under study. However, during their studies of the Auburn area, WCC identified two north-northwest-trending zones which have general structural continuity with branches of the Bear Mountains fault zone to the north and south.

These zones, termed lineaments, are locally 400 to 600 feet wide and exhibit "aligned linear elements" which are "...generally coincident with zones of Mesozoic deformation..." within the metamorphic bedrock. They include the DeWitt - Salt Creek lineament located about 0.5-mile east of the RM 20.1 site, and the Pilot Hill - Maidu East - Deadman lineament zone passing about 800 feet west of the site.

As noted earlier, the Secretary of the Interior, in July 1979, announced the adoption of State recommended design parameters for Auburn Dam. With respect to the potential for foundation displacement at the Mile 20.1 site, the California State Geologist recommended a surface displacement of 9 inches "...as the design parameter with the

possibility of this movement taking place along a single fault surface or distributed among several".

It should be noted that the structural relationships of the Maidu East shear zone and faults in the foundations of the alternate damsites may be different than that of the RM 20.1 damsite, and subject to different displacement parameters.

Seismicity. - The damsites are located in a region of relatively low to moderate seismicity. Historically, occasional tremors have been felt in the Auburn area. The tremors, however, have resulted from distant earthquakes in regions of higher seismicity. Examples include the April 1906 San Francisco earthquake (Richter magnitude 8.25) located approximately 110 miles west of Auburn, and the September 1966 Truckee earthquake (magnitude 5.8) located approximately 65 miles east of Auburn.

Small to moderate earthquakes have occurred in the western foothills of the Sierra Nevada. Most seismic activity is concentrated in the Nevada City - Grass Valley area and the Oroville - Chico area. The largest earthquakes recorded since records have been kept (1850) were the 1940 Oroville event (magnitude 5.7) located approximately 18 miles north of Lake Oroville, and the August 1975 Oroville event (magnitude 5.7) located approximately 7 miles south of Oroville.

Geologic evidence gathered in the vicinity of Oroville Dam and the Auburn RM 20.1 damsite, following the August 1975 Oroville event, has established a precedence for considering the Foothills fault system to be active. Faults of the Foothills fault system within a 2-mile radius of the Auburn damsites are considered to be capable of generating a MCE of magnitude 6.5

MCE's from areas having high seismicity outside the area of the Auburn damsites range from magnitude 8.5 within the Coast Ranges 100 miles west of Auburn, to magnitude 6.5 25 miles north near Nevada City. These sources would not impose seismic ground motions as great as the MCE (magnitude 6.5) generated from the Foothills fault system in the vicinity of the damsites.

Seismic ground-motion parameters and an MCE of magnitude 6.5 were established for the RM 20.1 damsite by the Secretary of the Interior in July 1979. Those parameters can probably be interpolated to the other alternative damsites because of their proximity to the RM 20.1 damsite.

Site Geology

The following discussion of the site geology is taken directly from the USBR's reports titled "Auburn Dam Excavation and Treatment, Records of Geologic Investigations, Part I of IV" (10), "Project Geology Report, Seismic Evaluation of Auburn Damsite, Summary Volume" (14), and "Design and Analysis of Auburn Dam, Vol. One" (16).

About 98 percent of the surface is covered with some type of surficial material: colluvium, landslides, or river alluvium. The

largest areas of rock exposure are in the river channel and the lower canyon slopes.

The major component of colluvium is slopewash, which usually ranges from 0.5 to 10 feet thick, averages about 2 feet thick, and consists of weathered rock fragments in a loose matrix of silty soil. This material forms on slopes subject to mass wasting and generally overlies 1 to 4 feet of rock affected by creep. Residual soils are poorly developed and found only on the flatter slopes, such as the upper left abutment of RM 20.1 and the upper right abutments of RM 19.2 and 19.0, where they are 2 to 3 feet thick.

Units designated as landslides are very old rubble slides that have attained a state of equilibrium through natural adjustment of the original slide mass. They mostly range in thickness from about 10 to 40 feet. Many of the landslides are bounded on at least one side by a continuous planar structure such as a shear zone or fault. Landslides may be reactivated by undercutting during construction activities, particularly during wet weather.

Amphibolite is the predominant and most competent rock type throughout the area. Where unweathered it is a hard dense rock. This fine-grained feldspar-amphibole schist is derived from low-grade regional metamorphism of Mesozoic volcanic flows and tuffs. Accessory minerals present in this rock are chlorite, epidote, carbonate, and quartz. The attitude of the foliation is relatively uniform throughout the area, with an average strike of about N.30°W. and an average dip of about 75°NE.

Locally within the amphibolite are metasedimentary and metavolcanic rocks of varying composition. The metasedimentary rocks range from laminated metashale and thin-bedded metasandstone, which are softer and more highly foliated than the amphibolite, to harder highly siliceous metachert and quartzite.

Throughout the area are numerous fine grained and porphyritic veins and dikes. The dikes occur throughout the area both as single isolated units and as local swarms where they are spaced 5 to 20 feet apart. They dip mostly from 5° to 30° to the southwest. Dike contacts exhibit a variety of conditions. Most are planar but irregular with small steps interlocking them with the country rock. Sheared contacts generally are planar and smooth.

Veins of quartz and calcite occur mostly along joints and shear zones. Veins along joints usually are less than 0.5-inch-thick. Those along shear zones generally are 0.1 to 0.3-foot-thick, with local pods to 5 feet thick.

Talc and chloritic rocks are of special importance because they are the weakest rock units in the area. Individual zones or complexes of these units, including serpentine, are referred to as T-zones. T-zones occur as tabular zones parallel to the metamorphic foliation and as large discordant masses which are largely serpentine. In the foundation excavation for RM 20.1 the

The T-zones were reported to range from 0.1-foot to several hundred feet in thickness in the construction area. This included areas outside the foundation (upper left abutment). In the foundation area only T-0 is greater than 45 feet. Elsewhere in the foundation T-14 ranges from 25 to 40 feet in width (mid-right abutment). Two discontinuous T-zones, T-3 and T-18 which die out downstream of the foundation, have widths ranging from 4 to 20 feet, and 5- to 23 feet in thickness, respectively. Most T-zones range in thickness from 0.1-foot to 5 feet, with the five major T-zones having average thicknesses ranging from 2 to 30 feet. Generally, they are discontinuous and commonly bifurcate or braid through the country rock. Portions of these zones may be sheared and contain varying percentages of soft gouge.

Discontinuities occur throughout the area in the form of shear zones, faults, joint sets, and cleavage planes.

Shear zones, which are relatively planar zones of fragmented rock, usually contain some clay gouge. Shear zones which parallel the foliation within the T-zones can be somewhat more continuous, on the order of several hundred feet. Most shear zones crosscut the foliation and dip to the southwest. They range from single thin layers of clay gouge, to zones 5 to 20 feet thick containing various amounts of clay gouge, quartz-calcite veins, and dikes They are limited in length from a few hundred feet to more than 1,000 feet. Shear zones which have been interpreted as being continuous over large portions of the area have been termed faults, or F-zones. The majority of these faults dip moderately to the southwest. They consist of one or more shear zones composed of highly variable amounts of clay gouge, rock fragments, and quartz-calcite veinlets.

At the RM 20.1 site F-zones and T-zones constitute more or less continuous planes of weakness that form boundaries of large blocks within the foundation. Although nine joint sets were identified in the early stages of investigations for the thin-arch dam, the faults seem to have developed parallel to two major joint sets, N.40°W. with a 45°SW dip, and N.60°W. with a 15 to 20°SW dip. Although the identification of these joint relationships for the most part were developed in the foundation excavation for the thin-arch dam, it is highly probable that the same or similar joint sets continue southward into the RM 19.2 and 19.0 sites.

Site Geologic Analysis

<u>River Mile 22.1 Damsite</u>. -

<u>Investigations</u>. - There have been no explorations above the streambed on the left abutment at this site. All previous exploratory holes were drilled in the streambed and on the right abutment a few hundred feet upstream of the RM 22.1 site proposed by Bechtel. Explorations at this site were halted when it was determined that a large landslide existed on the right abutment. Further explorations would be required to determine feasibility of this site.

Spillway. - The spillway for an embankment alternative at RM 22.1 would be located on the ridge which forms the left abutment. The spillway design for the large dam shown in the Bechtel report is of major concern because it cuts diagonally across the toe of a large slide located on the east side of the river at approximately RM 21.6. The spillway would be excavated into metavolcanic and metasedimentary rock. The spillway would probably require lining. Explorations would be needed for final determination.

Tunnel and Outlet Works. - The tunnel and associated outlet works excavated in the left abutment, would be founded in metavolcanic and metasedimentary rock like that in the spillway. The degree of fracturing and possible presence of shears or faults along the proposed alignment is uncertain and would require an extensive exploration program to determine.

Landslides. - The large landslide mass located upstream of the right abutment of the RM 22.1 site is a feature which underlies a portion of Highway 49, as well as several buildings located above the highway. In addition, a large spillway located on the spillway alignment downstream of the dam poses a threat of slope failure into the spillway.

If a small dry dam concept is adopted, the relatively rapid reservoir filling and drawdown increases the possibility of failure of the slide upstream of the dam. This was demonstrated by the numerous landslides which occurred in the flood and the subsequent failure of the cofferdam at the RM 20.1 site. The removal and or stabilization of the slide mass should be addressed in the consideration for the dry dam concept. If the staged concept is adopted, the same concerns remain and should be addressed here also. The large dam concept presents less of a problem since Highway 49 and the existing buildings would have to be relocated or removed prior to inundation of the reservoir. However, the effect of a slide on a diversion tunnel must still be considered.

The Environmental Impact Statement addressed the consequences of a landslide mass on a 2.3 million acre foot reservoir and concluded that adequate freeboard was designed into the dam to absorb any wave generated by the slide. But should a dry dam, or small dam concept be adopted the possibility of a large slide damming the river and creating essentially an ungated structure will need to be considered.

In addition, numerous smaller slides exist in the canyon which would be affected similarly but with fewer and less serious consequences. The decision to remove or stabilize each of these slides would be dependent upon the location and size of the dam configuration selected.

Dam Foundation. - Based on information prepared by Mr. E.C. Marliave (2), the foundation materials at this site consist primarily of amphibolite schist with scattered areas of phyllite. The rock is weathered and fractured and a significant amount of chloritized or serpentinized schist is located below the site on the right side of

the river. Quartz, found in shear and fracture systems and healing the original fractures, has been subsequently fractured in most instances. Schistosity generally strikes at right angles to the stream at this location. The dip of the schistosity varies by about 30 degrees on either side of vertical and in some areas, dips downstream. Numerous joint sets have been developed, some of which are unfavorable to the site. Two sets dip steeply downstream, three sets dip gently downstream and three sets dip from each abutment toward the channel. All of these joint sets could be the source of problems during construction and grouting of a dam at this site.

Treatment in the form of shaping of the nearly vertical rock face at the base of the left abutment would be required for both an embankment and RCC dam.

Grouting of the dam foundation to reduce seepage may have to be extended well outside the extent of the dam foundation on the left abutment because of the thin ridge which makes up that abutment. Explorations and water pressure testing would be required to ascertain the permeability of the materials in the ridge and to determine the amount of seepage which would be acceptable through the ridge without endangering the integrity of the structure.

Faults and Seismicity. - Only a small amount of geologic mapping was conducted at this site before it was eliminated from consideration. Mapping showed a narrow sheared serpentine zone traversing the canyon where the upstream toe of an embankment dam would be located. In addition, there is a large serpentine body on the right abutment which passes through the area of the downstream abutment interface. The relationship of these zones to those at the RM 20.1 site is unknown, and would require extensive mapping to identify. E.C. Marliave (2) reported that the river at this site follows a wide shear zone up to 200 feet in width. He also noted the presence of a small fault exposed in the old railroad cut on the right abutment at approximately elevation 750 feet. In addition, numerous shear zones were noted in various places. Although no faults were found during the mapping of this site, it is in closer proximity to the Salt Creek lineament than any of the other sites. A site-specific fault evaluation may be required.

Site Access. - There is presently no access to the left abutment. Access to the right abutment is limited to Highway 49 which traverses high on the abutment, and an abandoned narrow gauge railroad grade low on the abutment.

Site Clearing. - No clearing has been accomplished at this site. Overburden on the left abutment appears to be relatively thin with numerous rock outcrops. The right abutment contains areas of slide material and talus. Drill hole information upstream from the dam axis indicates up to 150 feet of material would need to be removed.

Location of Construction Areas. - Due to the steep canyon walls, the site provides poor access with a limited work area. The most feasible spot for a construction area is probably high on the

right abutment in the area of the existing California Parks and Recreation Department buildings on Highway 49.

River Mile 20.1 Damsite. -

Investigations. - Explorations for the RM 20.1 site are extensive and thoroughly investigate all features at the site. However, the relocation of the dam axis may require additional explorations but it is expected that these would be minor.

Spillway. - The spillway design for the thin-arch dam would not be used on any of the alternative concepts proposed by Bechtel. The concrete dam alternatives have their spillways incorporated into the face of the dam. An embankment dam would require a stepped spillway approximately 4,500 feet in length cut through the ridge which forms the left abutment. Excavation would be primarily in amphibolite with narrow zones of metasedimentary rock and serpentine. The downstream end of the spillway would cut the toe of a slide located on the east side of the canyon at approximately RM 19.5.

Tunnel and Outlet Works. - The rolled concrete and concrete gravity alternative concepts could use the existing diversion tunnel without modification. Construction of an embankment dam would require modification of the existing tunnel to add approximately 1,400 feet to the downstream end. The outlet works as conceptualized by Bechtel would include two conduits in the diversion tunnel, combined with a tunnel to a hydroelectric generating plant.

Landslides. - Over thirty small to large landslides occurred during construction of access roads and foundation excavation for the thin-arch dam. The largest landslide stabilized during construction is slide 21 (estimated volume 400,000 cubic yards (CY)). Removal and resloping which included a subsidiary slide 25 (estimated 50,000 CY) required excavation of 1,080,000 CY. The slide material was removed and placed as engineered fill in three ravines downstream of the left abutment. If an embankment dam is constructed at RM 20.1 site, additional explorations will be needed to determine whether the fill material in the ravines could be incorporated into the design of the dam. If tests show that the material is unusable, it will require removal.

Three known landslides have the potential to cause a hazard to the alternative dams at this site. One (slide 16) is located just downstream of the right abutment keyway for the thin-arch dam. Based on data supplied from drilling and inclinometer readings, this slide is fairly well delineated at 1.2 million CY. A few hundred cubic yards of the slide was removed in October 1975 where it sloughed into one of the contractor's work areas. It would form a portion of the right abutment foundation of the embankment dam alternative and would require removal.

Two slides located on opposite sides of the river downstream of RM 19.5 lie outside of the foundation footprint for any of the dam alternatives, but could cause a hazard for appurtenances for an embankment dam. The slide on the left (east) canyon wall is the

largest in surface area and, as discussed earlier, would be located at the toe of the spillway for the embankment alternative. In addition, the downstream portal for the lengthened diversion tunnel would be located at the toe of the slide. The slide material would require either removal or slope stabilization by conservative cut-slopes.

Dam Foundation. - As discussed earlier, any additional explorations needed for construction of a dam at this site are limited to a few drill holes required to investigate the geologic conditions of the diversion tunnel extension and spillway alignment for the embankment dam alternative. It is felt that no additional explorations would be required for the concrete gravity alternative dams.

Faults and Seismicity. - The faults which were identified and the seismic parameters that were developed for the thin-arch dam, and later for the CG-3 configuration, would be applicable for the design of a dam at this site.

Site Access. - Although portions of the approximately 12 miles of access and haul roads built for use during construction of the thin-arch dam will require repair or replacement, many are still in use or are usable at the present time.

Site Clearing. - Much of the foundation area for the alternative dams at the site have already been cleared for construction of the thin-arch dam and slope stabilization has been completed. Depending on the configuration of the final alternative dam design, the amount of clearing of vegetation and overburden will vary.

Location of Construction Areas. - The construction areas developed for construction of the thin-arch dam can be used for any alternative dam at the site.

River Mile 19.2 Damsite. -

Investigations. - Investigations for this site include both exploratory drill holes and trenching. Thirty exploration drill holes were completed at this site, and approximately 15,000 feet of exploratory trenches were mapped. No exploration or test tunnels were excavated and no rock mechanics testing was performed.

Spillway. - The spillway for an embankment dam constructed at this site could be located on the right abutment. The bedrock here is quartz diorite which varies from highly weathered to fresh. For a small dam, the spillway could be located across a saddle through which the existing construction road passes. A large dam would require locating the spillway further up the hill and would result in more required excavation. Bechtel (47) considered a spillway design which would be approximately 255 feet wide and 1200 feet long excavated approximately 120 feet deep into the quartz diorite. If an expandable dam concept were adopted, raising the embankment would require filling the spillway of the smaller dam and constructing another spillway.

Diversion Tunnel and Outlet Works. - A diversion tunnel constructed through the right abutment would pass through serpentine in the upstream portion of the tunnel and amphibolite schist on the downstream end. The need for tunnel support must be assumed in the serpentine and probably in the amphibolite. Excavation for the portals is expected to require deep cuts and slope stabilization. A possible alternative to the right abutment tunnel at this site would be to extend the existing diversion tunnel an additional 3000± feet by means of a conduit. Bechtel conceptualized the outlet works for the alternative designs as consisting of two 72-inch gated conduits in a new 33-foot-diameter diversion tunnel for an embankment dam and two 72-inch-diameter pipes incorporated within the dam for the RCC design. The outlet works for the RCC dam are similar to those designed by the USBR for the CG-3 dam at mile 20.1.

Landslides. - Exploratory drill holes on the left abutment revealed the presence of a deep block slide located between the axis of the dam and Knickerbocker Creek. Excavation to depths of about 120 feet would be required to remove this slide debris from the dam foundation. In addition, a large, shallow slide is located upstream of the axis on the left abutment, and a small slide is located upstream of the axis on the right abutment.

Dam Foundation. - Preparation of the dam foundation would include removal of both slides located on the left abutment. One of these is a deep-seated slide which would require excavation of about 120 feet of slide debris. The foundation material on the left abutment consists mainly of amphibolite schist with small amounts of metasediments high on the abutment. The right abutment is underlain by serpentine and amphibolite schist. Weathering is moderately deep with open, closely spaced joints, heavy iron oxide staining, and disintegration to an average depth of approximately 30 feet with a maximum depth of 50 feet. The rock is cut with numerous structures including small faults, shear zones, dikes, quartz veins, and variable joint patterns. Excavation of approximately 30 feet on the abutments and 15 feet in the river channel would be required in the cut-off portion of an embankment dam. An average excavation of about 5 feet would be required over the rest of the foundation.

Foundation grouting for a dam at this site would require a grout curtain about 200 feet in depth. Moderately high grout takes are expected to depths of about 150 feet.

Faults and Seismicity. - A 5 to 15-foot-wide, near-vertical fault cuts diagonally across the dam axis in the channel area at RM 19. The fault as mapped by USBR is approximately 3,600 feet long. It extends north-northwest from the Oregon Bar pluton approximately 2,800 feet to the river channel, where it splays into several shears and finally disappears under the large landslide on the east side of the canyon at RM 19.4. The USBR has also mapped a shear on the canyon wall northwest of the landslide which may be an extension of this fault. The seismic studies performed for the RM 20.1 site encompass this area and the results of that study are applicable here also.

Site Access. - The present access to the left abutment consists of a narrow road with numerous switchbacks which were originally excavated as exploratory dozer trenches for preconstruction investigations conducted at the original USBR embankment site. Portions of the road were washed out during the rains of 1986 and remain impassable. Access to the right abutment consists of the road designated on maps as the power plant access road.

Site Clearing. - No clearing has been accomplished at this site. It is expected that removal of the organic soil and root zone will require excavation of about 2 feet of material.

Location of Construction Areas. - This site is located in a narrow portion of the canyon with limited work area. It is expected that the construction work area used during work at the RM 20.1 site would provide space required.

Cofferdams. - Construction of an embankment dam at this site would necessitate rebuilding the cofferdam upstream from the site. Since the site is within the upper reaches of Folsom Lake, a downstream cofferdam may be needed to protect the construction site from backwater from the lake.

Knickerbocker Canyon Diversion. - Diversion of water from Knickerbocker Canyon will be required for an embankment dam at this site.. Flows through the canyon would impinge upon the downstream portion of the embankment.

River Mile 19.0 Damsite. -

Investigations. - Investigations for the site include 30 drill holes and approximately 15,500 feet of exploratory trenches. The majority of this work was performed outside the immediate area of the proposed axis for the dam. No explorations were done on the left abutment for the area downstream from the proposed axis.

Spillway. - The spillway for a small embankment dam would be located through the saddle on the right abutment. For a large embankment dam, the spillway would be located higher on the right abutment and would require more excavation to build. Both sites would be founded on quartz diorite which is variably weathered. The average depth to firm rock is 40 feet. A large quantity of the material which would be excavated from the spillway would be usable in an embankment dam. An expandable embankment dam at this site would require filling of the spillway for the smaller dam and construction of a new spillway like that required at RM 19.2.

Tunnel and Outlet Works. - The diversion tunnel and outlet works for an embankment dam at the site would be located through the right abutment. The upstream portal and several hundred feet of the upstream end of the tunnel would be founded in serpentine with some soft talc and/or amphibolite. Excavation for the portal would require deep cuts and stabilization of the cut-slopes. In the tunnel section, support must be assumed. The remainder of the tunnel, the downstream portal, and the outlet works would be founded on granitic rock.

Support in the granitic portion of the tunnel is assumed for a portion of its length. In the excavation for the portals, firm rock is estimated to be about 25 feet deep and for the outlet works, 50 feet to fresh rock.

Landslides. - The known landslide masses associated with this site are relatively shallow and mostly small in area. The largest slide is located upstream of the axis on the left abutment. The right abutment has several small slides, one of which is located on the axis alignment. Removal of all or parts of these slides would be required. The materials removed may be partially usable in an embankment structure.

Dam Foundation. - Preparation of the dam foundation would include removal of slide material located within the cutoff area and any other material within the footprint of the dam necessary to provide a suitable foundation. On the left abutment, the foundation rock is primarily amphibolite with minor amounts of metasediments high on the abutments. The lower portion of the right abutment is amphibolite and the upper right abutment is quartz diorite. The contact between these rock types is near the saddle proposed for a possible spillway location. Weathering is moderately deep with disintegration to an average depth about 30 feet, excluding areas of slides. Joints are closely spaced, open, and heavily iron oxide stained. The rock contains small faults and shears, dikes, quartz veins, and variable joint patterns. Excavation within the cut-off portion of an embankment dam would be approximately 30 feet on the abutments and 15 feet in the river channel. Excavation over the remainder of the foundation would average about 5 feet.

Foundation grouting for a dam at this site would require a grout curtain about 200 feet deep. Moderately high grout takes are expected to depths of about 150 feet.

Faults and Seismicity. - This site is positioned more nearly parallel to the regional structure than the other three sites. Two faults pass through the saddle on the right abutment. No other faults are known to exist at this site but this does not preclude their existence. The seismic studies performed for the RM 20.1 site encompass this area and the results of that study are applicable here also.

Site Access. - There has been no access development to the left abutment at this site. Access to the right abutment consists of the road used for access to the RM 20.1 site, the power plant access road,and a road on the abutment used for access to the river at Oregon Bar.

Site Clearing. - No clearing has been done at this site. Removal of organic soil and the root zone is expected to require excavation of about 2 feet of material.

Location of Construction Areas. - Similar to the RM 19.2 site, this site is located in a narrow portion of the canyon with

limited work area. It is expected that the area at approximately RM 20.0 would provide space needed for construction activities.

<u>Cofferdams</u>. - Construction of an embankment dam at this site would necessitate building cofferdams on both the upstream and downstream sides of the construction area. Since the site is within the upper reaches of Folsom Lake, the downstream cofferdam would be needed to protect the construction site from backwater from the lake.

<u>Knickerbocker Canyon Diversion</u>. - A permanent structure for the diversion of water from Knickerbocker Canyon would be required for construction of an embankment dam at the RM 19.0 site. It is expected that diversion of water from the canyon would not be required if a concrete dam were constructed because the dam would lie downstream of the mouth of the canyon.

ADVANTAGES AND DISADVANTAGES

River Mile 22.1 Damsite

<u>Advantages</u>. -

1. Requires the least amount of construction materials of the four sites.

2. Diversion tunnel through left abutment ridge would be straight and relatively short.

3. Site is closer to the upstream aggregate sources.

4. Spillway on left abutment would be relatively short.

5. No downstream cofferdam required.

<u>Disadvantages</u>. -

1. There is a large landslide on right side of canyon which would need to be either removed or stabilized.

2. There is no economical location for a spillway for a low embankment dam.

3. A spillway for a large embankment dam would cut the toe of a large slide on the left canyon wall downstream of the dam. This would present a possible slope stability problem.

4. The reservoir from a large dam would back up and flood the PCWA's Oxbow power plant. Also, this reservoir would extend into the portions of the Upper American River designated as "wild and scenic".

5. The very steep left abutment would require shaping by drilling and blasting and make placement of an embankment difficult.

6. There are no access roads on the left abutment, and limited access to the right abutment.

7. There are few sites for the location of construction facilities.

8. Highway 49 would have to be relocated prior to initiation of construction.

9. The site would require an extensive exploration program and fault evaluation for knowledge of foundation conditions. This may require the reevaluation of seismic parameters due to the distance from the RM 20.1 seismic studies.

10. The thin ridge which forms the left abutment is comprised of fractured rock which may require special grouting to prevent seepage.

11. Will necessitate restoration of the RM 20.1 site downstream (32).

River Mile 20.1 Damsite

Advantages. -

1. Explorations, for the most part, have been completed.

2. The geologic conditions are well known.

3. Construction access roads and access facilities presently exist, although are in need of repair.

4. Adequate contractor use areas presently exist.

5. A diversion tunnel was completed for the USBR concrete arch dam previously designed, however, it may require modification for any future construction.

6. Seismic studies have been completed.

7. Seismic parameters for fault displacement within the foundation assigned to the thin-arch dam previously under construction apply to any one of the alternative dam designs at this site.

Disadvantages. -

1. Requires the largest volume of construction materials and therefore is more costly than other sites.

2. The site for a spillway for a low embankment dam is poor due to the amount of excavation that would be required on the left abutment.

3. The upstream cofferdam will need to be rebuilt if an embankment dam were selected at this site.

4. A landslide on the right abutment would need to be removed if an embankment dam were chosen.

River Mile 19.2 Damsite

Advantages. -

1. The geology and seismicity of the site are fairly well known

2. Previously used construction areas from the RM 20.1 site may be used at this site.

3. The upstream existing diversion tunnel at the RM 20.1 site can be utilized.

4. The quartz diorite bedrock would provide a good spillway foundation.

5. The dam would require a somewhat smaller volume of construction material than a dam at RM 20.1 site.

Disadvantages. -

1. Existing diversion tunnel upstream would need to be either lengthened or a new one built for an embankment dam. If a new one was built, it would require tunnel supports in the fractured rock.

2. A downstream cofferdam would be required, except perhaps for an RCC dam.

3. A spillway located on the right abutment would require a large excavation.

4. An embankment dam alternative would require a diversion facility for stream flows from Knickerbocker Canyon.

5. Much additional explorations would be required.

6. A deep landslide on the left abutment would need to be removed.

7. Serpentine bedrock in the right abutment could present stability problems during construction.

8. No site-specific fault studies have been performed.

9. The proposed powerplant may be inundated by Folsom Lake.

River Mile 19.0 Damsite. -

Advantages. -

1. A dam at this site would require a smaller volume of construction material than a dam at the RM 20.1 site.

2. A relatively short diversion tunnel would be required for an embankment dam.

3. Quartz diorite bedrock in the right abutment would provide good tunnel material requiring a minimal amount of tunnel support if a concept other than RCC were chosen.

4. The spillway for an embankment dam would be founded largely in the quartz diorite of the Oregon Bar pluton which would provide good foundation conditions.

5. The construction areas developed for the RM 20.1 site may be used at this site.

Disadvantages. -

1. The site would require the construction of a new diversion tunnel.

2. A downstream cofferdam would be required, except perhaps for an RCC dam.

3. A spillway on the right abutment for an embankment dam would require a large excavation.

4. An embankment alternative would require a diversion facility for stream flows from Knickerbocker Canyon.

5. The site would require an extensive exploration program for knowledge of foundation conditions.

6. Site-specific fault studies would be required in the foundation area.

7. There is no existing access to the left abutment area.

8. The proposed powerplant may be inundated by Folsom Lake.

CONCLUSIONS and RECOMMENDATIONS

River Mile 22.1 Damsite

Recommend the RM 22.1 site be eliminated from further consideration.

River Mile 20.1 Damsite

In the opinion of the Geology Section the RM 20.1 site should be considered the most feasible damsite for construction **if** time restraints become the deciding factor for site determination.

According to the USBR reports, this site was considered to be the most feasible in overall cost because of the extensive surface and subsurface explorations and testing completed, the existing in-place facilities, knowledge of foundation conditions, and the minimal foundation preparation required. A dominant factor in the decision to build a dam at this site over any of the other sites is the amount of time to be saved by utilizing the existing information and foundation work that has already been completed. The geology at the site has been investigated to the extent that only minor amounts of explorations would be needed prior to the initiation of construction.

Mr. Wendel Carlson (retired former Auburn Dam Project Geologist, USBR) estimated that a savings of 18 to 24 months could realized by constructing a dam at RM 20.1 as opposed to 19.0.

River Mile 19.2 Damsite

Recommend the RM 19.2 site be eliminated from further consideration.

River Mile 19.0 Damsite

The RM 19.0 site should not be eliminated from consideration unless completion time becomes the deciding factor on selection of a damsite.
At this time there is insufficient geologic information about the site to determine whether it is more or less feasible than the RM 20.1 site.

Based on the USBR estimates, an RCC dam at RM 19.0 would require approximately 30 percent less concrete for construction than an RCC dam of similar height at RM 20.1. But, as pointed out by Mr. Carlson, geologist for the USBR (37), the amount of excavation which would be required to prepare the RM 19.0 site reduces the cost advantage to about 25 percent lower cost for an RCC dam at RM 19.0. Adding the cost of site preparation, construction of access roads, and a diversion tunnel to consideration of the RM 19.0 site reduces the cost savings of the RM 19.0 site to about 5 percent or approximately $47 million in 1986 dollars.

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APPENDIX M-5-A

FOUNDATION PROPERTIES AT THE RM 20.1 DAMSITE

In the initial phases of design, the lack of test data required the designers to assume that bedrock at the site was a homogeneous isotropic rock mass. They started design work by considering a uniform value for the modulus of deformation of 2,500,000 lb/in^2 throughout the foundation.

Intermediate geologic and laboratory examination of the rock from the foundation indicated the deformation modulus was nonhomogeneous across the site. The interim moduli listed in the following table assumes the modulus is isotropic.

Elevation ft (m)	Left Abutment (10^6lb/in^2)	Right Abutment (10^6lb/in^2)
1135 (345)	1.80	2.00
1050 (320)	1.80	2.00
950 (290)	1.80	2.00
900 (275)	2.15	1.75
850 (260)	2.50	1.50
750 (230)	2.50	3.00
650 (200)	3.00	3.40
550 (170)	3.00	3.80
485 (150)	3.00	3.80
460 (140)	2.50	2.50
450 (135)	2.50	2.50

The following tables show the average deformation moduli from the in-situ radial and uniaxial jacking tests performed in the tunnels excavated in the foundation for the thin-arch dam at the RM 20.1 site. It should be noted that the testing was done only on the metasedimentary and metavolcanic rock types in the foundation. Quartz diorite, which comprises the right abutment of the RM 19.0 and 19.2 sites is not present at the RM 20.1 site, and therefore the foundation properties of that type of rock are not known.

Table 1 shows the average deformation moduli from the three in-situ radial jacking tests, and Table 2 shows actual deformation values obtained from the in-situ uniaxial jacking tests.

TABLE 1

RESULTS OF IN-SITU RADIAL JACKING TESTS

Rock Type Tested	Average Deformation Modulus (lb/in^2) 1.5 to 20.0-foot depth	0 to 1.5- foot depth
Metasedimentary rock	3.86×10^6	2.81×10^6
Slightly weathered amphibolite	1.79×10^6	1.75×10^6
Amphibolite with metagabbro	10.02×10^6	7.60×10^6

TABLE 2

RESULTS OF IN-SITU UNIAXIAL JACKING TESTS

Test No.	Rock type tested	Deformation modulus (lb/in^2) Anchor 1 to anchor 7	Deformation modulus (lb/in^2) Anchor 1 to sensor head	Remarks
U1A1-1	Fresh talc schist	2.04×10^6	1.35×10^6	
U1A1-2	Fresh talc schist	1.65×10^6	0.91×10^6	
U1A3-2	Lightly weathered metasediment	0.49×10^6	0.52×10^6	Maximum load to 600 lb/in^2
U1A4-2	Amphibolite	2.37×10^6	2.19×10^6	
U1B1-1	Metasediment	0.86×10^6	0.74×10^6	
U1B1-2	Metasediment	0.84×10^6	0.42×10^6	
U1B2-1	Chlorite schist and metasediment	0.20×10^6	0.17×10^6	
U1B2-2	Chlorite schist and metasediment	0.33×10^6	0.32×10^6	
U2B1-1	Amphibolite and dike	8.09×10^6	4.71×10^6	
U2B1-2	Metagabbro	10.52×10^6	9.31×10^6	
U2C1-1	Amphibolite	*	6.50×10^6	*Defect in concrete pad prevented calculation
U2C1-2	Amphibolite with metagabbro	4.80×10^6	4.12×10^6	
U2C3-1	Amphibolite	9.20×10^6	6.15×10^6	
U2C3-2	Amphibolite	12.33×10^6	1.41×10^6	
U3A1-1	Talcose serpentine	2.13×10^6	1.07×10^6	
U3A1-2	Talcose serpentine	1.54×10^6	1.59×10^6	
U3A2-1	Metasediment	*	2.64×10^6	*Malfunction of anchor 7
U3A2-2	Metasediment	3.57×10^6	1.99×10^6	
U4A2-1	Foliated amphibolite C	1.81×10^6	1.21×10^6	
U4A2-2	Foliated amphibolite C	2.82×10^6	1.19×10^6	
U4B1-1	Fresh amphibolite C	19.18×10^6	5.93×10^6	
U4B1-2	Fresh amphibolite C	7.03×10^6	3.98×10^6	
U4B2-1	Fresh amphibolite C	5.53×10^6	5.27×10^6	
U4B2-2	Fresh amphibolite C	7.00×10^6	2.03×10^6	
U5A1-1	Lightly weathered amphibolite C	1.27×10^6	1.66×10^6	
U5A1-2	Fresh amphibolite C	1.33×10^6	1.34×10^6	
U5A2-1	Weathered amphibolite	3.41×10^6	2.32×10^6	The foundation in the test area had been grouted prior to the test.
U5B1-1	Fresh amphibolite B and C	6.24×10^6	4.34×10^6	
U5B1-2	Amphibolite C	3.25×10^6	3.37×10^6	
U5B2-1	Chlorite schist C	4.16×10^6	0.95×10^6	
U5B2-2	Chlorite schist C	4.13×10^6	0.95×10^6	
U6-L	Serpentine	5.00×10^6	4.03×10^6	
U6-R	Serpentine	5.57×10^6	2.60×10^6	

Note: The letter designation following some rock types denote rock of differing composition.

The USBR also conducted in-situ plate gouge tests to determine the modulus of deformation across nine of the F-zones and T-zones encountered in the tunnels. The following table shows the results from that testing procedure.

TABLE 3

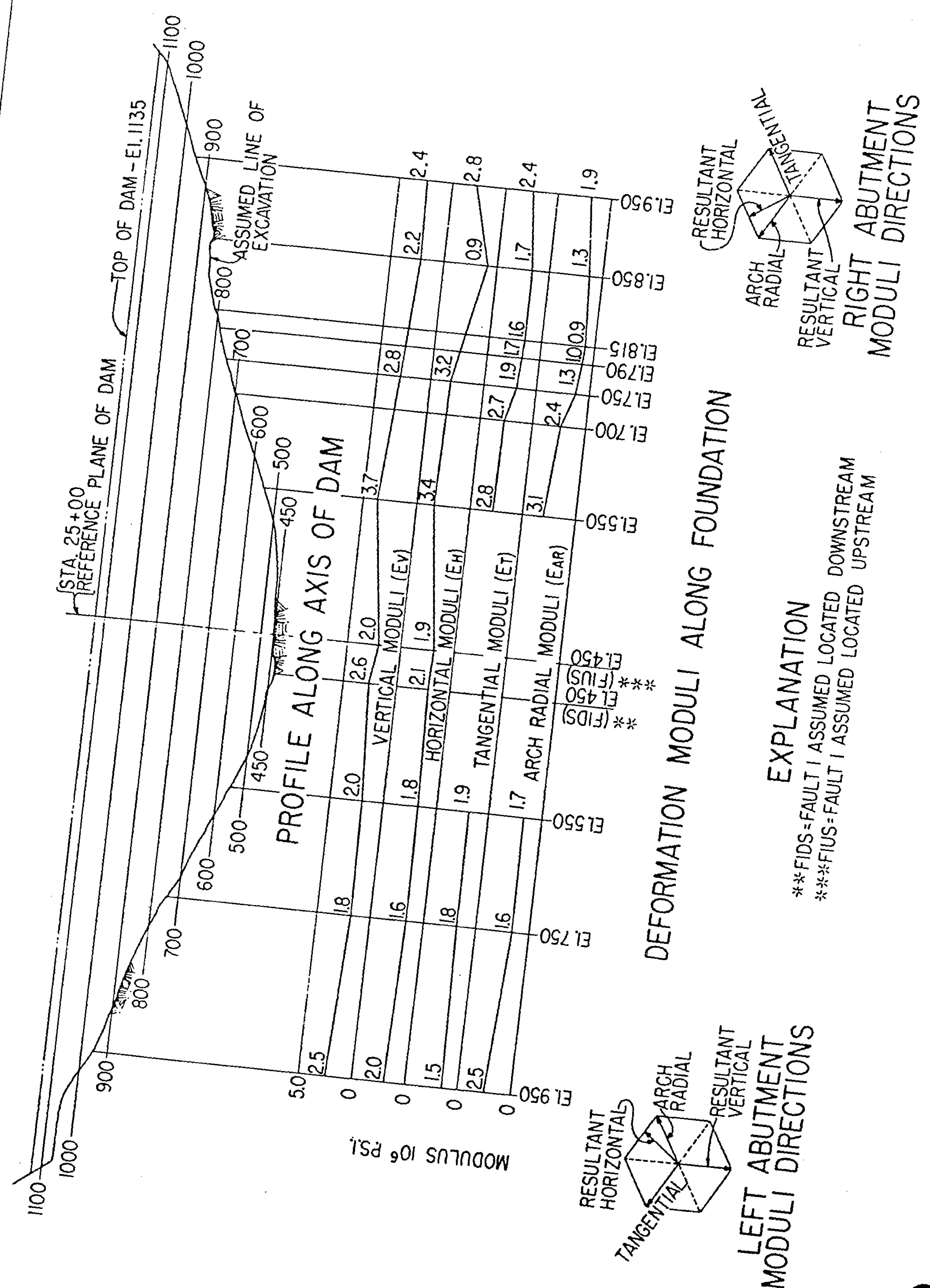
RESULTS OF IN-SITU PLATE GOUGE TESTS

Test No.	Fault, shear, or talc zone	Maximum load lb/in^2	Deformation modulus lb/in^2	Remarks
1-10+12	F-17	1,000	39,700	F-17 is a sheared dike, highly
1-10+12	F-17	1,000	136,200	fractured and contains gouge.
1B-3+04	F-16	1,000	62,000	F-16 is a shear containing gouge
1B-3+04	F-16	1,000	46,400	coated tack, chlorite schist B,
				and chlorite schist C fragments.
2-8+57	F-8	800	424,500	F-8 is a shear in chlorite schist
2-8+57	F-8	800	268,400	C; shear is highly fractured
				and contains gouge.
3B-2+90	F-1	1,000	5,300	F-1 at this location is a moist
3B-2+90	F-1	1,000	10,700	gouge with chlorite schist
3B-2+90	F-1	1,000	40,300	fragments.
4-6+75	F-1	1,000	136,400	F-1 at this location is a zoned
4-6+75	F-1	1,000	126,600	fault containing quartz and
4-6+75	F-1	1,000	191,000	calcite healing, and gouge seams.
4B-2+40	T-6	600	44,600	T-6 is a talc-serpentine zone
4B-2+40	T-6	600	44,600	having a shear containing moist
4B-2+40	T-6	600	115,500	talc fragments and gouge.
5-9+30	T-3	1,000	12,100	T-3 is a talc zone containing a
5-9+30	T-3	1,000	1,460	shear with gouge and talc schist
				fragments.
5A-2+79	F-1	800	55,700	F-1 at this location contains
5A-2+79	F-1	800	22,400	shears with gouge and broken
5A-2+79	F-1	800	36,500	slickensided chlorite schist C
				and B fragments.
5B-4+15	F-1	1,000	91,900	F-1 at this location contains
5B-4+15	F-1	1,000	100,200	shears with gouge and chlorite

Figure 1 taken from USBR's Desgin and Analysis of Auburn Dam report (16) shows the final multidirectional deformation moduli of the foundation rock across the foundation. They used these values in their Arch Dam Stress Analysis System (ADSAS) computer program to analyze the final design. That program utilizes a matrix solution in arriving at the proper division of load between vertical and horizontal elements.

Poisson's Ratio. For design purposes, the Poisson's ratio for all foundation rock is assumed to be 0.20.

Allowable Compressive Stress. The allowable compressive stress on the foundation rock was reported to be 1000 lb/in^2.



PROFILE ALONG AXIS OF DAM
DEFORMATION MODULI ALONG FOUNDATION
TOP OF DAM - EL.1135
STA. 25+00
REFERENCE PLANE OF DAM
ASSUMED LINE OF EXCAVATION
VERTICAL MODULI (EV)
HORIZONTAL MODULI (EH)
TANGENTIAL MODULI (ET)
ARCH RADIAL MODULI (EAR)
MODULUS 10⁶ PSI.
EL.950
EL.850
EL.815
EL.790
EL.750
EL.700
EL.550
EL.450
**(FIDS)
***(FIUS)
LEFT ABUTMENT MODULI DIRECTIONS
RIGHT ABUTMENT MODULI DIRECTIONS
RESULTANT HORIZONTAL
RESULTANT VERTICAL
ARCH RADIAL
TANGENTIAL
EXPLANATION
**FIDS=FAULT I ASSUMED LOCATED DOWNSTREAM
***FIUS=FAULT I ASSUMED LOCATED UPSTREAM

FIGURE 1

APPENDIX M-5-B

AUBURN DAM - GEOTECHNICAL, ENVIRONMENTAL, AND DESIGN REFERENCES

UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION (USBR)
REFERENCES

	Reference Location in Sacramento			
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AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 6

CONCRETE MATERIALS
AND
ROLLER COMPACTED DAM CONSIDERATIONS

JANUARY 1989

PREPARED BY

MATERIALS SECTION
GEOTECHNICAL BRANCH

OFFICE REPORT
CONCRETE MATERIALS AND
ROLLER COMPACTED CONCRETE DAM CONSIDERATIONS

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

TABLES

PLATES

OFFICE REPORT
CONCRETE MATERIALS AND
ROLLER COMPACTED CONCRETE DAM CONSIDERATIONS

AUTHORITY

This report is conducted under the authority of the Flood Control Act of 1962 (Public Law 87-874, dated October 23, 1962), and the 1987 Appropriations Act which directed the Corps to "engage in a one-year reconnaissance study of alternative means of flood control in the American River, California". This report was prepared by Materials Section, Geotechnical Branch.

PURPOSE

This report is intended to provide information on concrete materials, material properties, and design and construction considerations for a gravity dam near Auburn, California, on the North Fork of the American River. The sites initially evaluated include the mile 19.0, 19.2, 20.1, and 22.1 sites. The focus of this study was concentrated on the 20.1 site, the selected site for the dam alternatives, with a straight gravity dam and a reservoir capacity of 570,000 acre feet. Other alternatives with different storage capacities will be investigated. However, discussions for other sizes would be similar to discussions for the 570,000 acre feet size. Because other concurrent studies have focused on the 19.0 site as well, that site has been discussed in some detail in this report. This information will be used for preliminary structural analysis and cost estimating of the concrete alternatives evaluated in this feasibility study.

SCOPE

Since roller compacted concrete (RCC) has become the primary method of construction for all mass concrete gravity dams worldwide, this report will consider only a gravity dam constructed with RCC. RCC provides a concrete mass structure essentially identical to a conventional dam, and at far less cost. This report begins by providing a historical background of roller compacted concrete (RCC) dams. It then presents preliminary design considerations for an RCC gravity dam. An analysis of aggregate availability and evaluation of concrete properties for design follows. Then there is a discussion of RCC dam construction, and an analysis of some advantages of an RCC dam and expandability of RCC dams. Finally, a discussion of future effort for design of an RCC dam at Auburn is included.

PROJECT LOCATION AND BACKGROUND

The Auburn Dam project is located in northern California, on the North Fork of the American River, just east of Auburn, California. The dam sites being considered in this report are shown on Plate 1. The background of damsite selection by the US Bureau of Reclamation (USBR) and others is addressed in the "Geologic Review of Alternative Damsites - Auburn Dam Project"[1], prepared by Geology Section, Geotechnical

Branch. The following is a brief discussion of the recent developments related to concrete gravity dam proposals for an Auburn damsite.

USBR CG-3 Feasibility Report

Following the Oroville earthquake and subsequent re-analysis of the Auburn Dam Project, two new Feasibility Design Summaries were prepared by the USBR, one for a rockfill dam and one for a curved concrete gravity dam, both at the 20.1 site. The curved concrete gravity dam, called CG-3, was discussed in the August 1980 Feasibility Design Summary for Auburn Dam[2]. This report discussed geology and seismicity of the site and dam, concrete dam considerations, foundation preparation and grouting, spillway structures, and outlet and power works. This dam concept was basically a conventional concrete gravity dam that followed the footprint of the completed foundation preparation of the USBR arch dam design. The concrete in the dam was zoned by strength criteria from 3000 psi to 8000 psi, and had substantial amounts of very high strength concrete in the structure. The upstream face of this dam was vertical, and the downstream face was a constant slope at 0.68H:1V. The volume of concrete in the dam was approximately 9,700,000 cubic yards. Detailed seismic design analysis, including fault displacement, was described in this report.

Bechtel Evaluation of Auburn Dam Project

At the request of the California Department of Water Resources, Bechtel National, Inc., completed an evaluation of the Auburn Dam Project in November 1985[3]. This document covered a wide range of subjects relating to Auburn Dam, including several types of multi-purpose dams at four sites. The sites studied included the 20.1, 19.0, 19.2, and the 22.1 mile sites. The types of dams studied included rockfill, rockfill with concrete face, conventional concrete and RCC gravity. Cost analysis of the alternatives was the emphasis of this document. No structural analysis was performed in this investigation. Bechtel found the RCC gravity dam to be the least costly option, even though they used a very conservative section. Bechtel also discussed relocation of Highway 49 away from a concrete dam crest, and recommended further study of several dam and site combinations.

USBR Evaluation of Bechtel Report

USBR provided a response on a number of issues to the Bechtel Report in their report entitled "Evaluation of Auburn Dam Reformulation and Bechtel Report[4]. This included discussion of several damsites and dam types studied by Bechtel, staging (i.e., flood control dam expanded later to a multi-purpose dam), Highway 49 relocation, power aspects, and related issues. USBR disagreed with several of Bechtel's assumptions on RCC, including cost and placement rate. Information from various technical publications has shown that the low costs cited by Bechtel for RCC are accurate, and that very high placement rates are practicable. The USBR recommended a few changes to the RCC details, but agreed with the general concept as feasible. The USBR also provided a discussion of the advantages and disadvantages of each damsite.

HISTORICAL PERSPECTIVE OF RCC FOR DAMS

In order to provide additional insight into RCC for those not fully acquainted with this technology, a discussion follows on what RCC is, how it developed, some recent RCC dams, and RCC dams currently being designed and constructed.

What is Roller Compacted Concrete

The American Concrete Institute (ACI) Committee 207 Report on Roller Compacted Mass Concrete[5] defines "roller compaction" and "roller compacted concrete" as follows:

"Roller compaction: A process for compacting concrete using a roller, often a vibrating roller."

"Roller compacted concrete: Concrete compacted by roller compaction; concrete that in its unhardened state will support a roller while being compacted."

An important part of this definition is that RCC is concrete. RCC is not soil cement, nor is it a low quality concrete. RCC is merely concrete placed in an unconventional manner. The properties and design of RCC for mass concrete are similar to what was considered conventional mass concrete. RCC can now be considered the primary method for placing mass concrete for dams, since the majority of concrete dams are now being constructed with roller compaction.

Origins of Roller Compacted Concrete

Although the early origins of RCC are difficult to track precisely, at least two pavements are known to have been constructed with RCC in the 1940's. "RCC was developed as a result of efforts to design more economical concrete dams that could be constructed rapidly"[5]. The beginning of RCC for mass concrete is usually marked by a paper presented by Raphael at the Rapid Construction of Concrete Dams Conference in 1970, entitled "The Optimum Gravity Dam"[6], which presented the concept of using earthmoving equipment to place a cement-enriched embankment material for mass dams. Following this development, a number of public agencies and private industry began study, testing, design and construction of projects involving what was called "rollcrete" at that time.

In 1972, Cannon and the Tennessee Valley Authority (TVA) built a test section of RCC at Tims Ford Dam, and USACE constructed test sections at WES and at Lost Creek Dam, Oregon.

From 1974 to 1982, over 3.3 million cubic yards of RCC were used at Tarbela Dam, Pakistan, for erosion repair of the dam embankment and spillway erosion repair. In the late 1970's, two RCC projects were constructed by USACE in Alaska and Washington. USACE also designed Zintel Canyon Dam as an RCC gravity section during this period, but lack of funding prevented it's construction.

In 1982, the world's first all RCC dam was constructed at Willow Creek, Oregon, by USACE. This 169-foot high dam was constructed in less than 5 months, using more than 430,000 cubic yards of concrete.

Research on RCC in Japan was begun in 1974, culminating in the use of RCC as the body of Shimajigawa Dam in 1978, and for the foundation block in Ohkawa Dam in 1979. Use of RCC in several other dams in Japan has followed, typically using very thick lifts of 27.5 inches or greater.

In the late 1970's, extensive research in the United Kingdom was carried out on high flyash content RCC, resulting in test sections and a full-scale trial. Although this work did not result in dam construction, the results were used for the design and construction of Upper Stillwater Dam, Utah, by the USBR. This 296-foot high RCC dam was completed in 1987, using 1,470,000 cubic yards of concrete.

RCC has also been used for overtopping protection for embankments and as spillway structures.

Recent Roller Compacted Concrete Dams

"Since these first projects, RCC has rapidly gained popularity and it has been used in a number of completed structures in Brazil, Venezuela, France, Australia, and South Africa, as well as in the United States and Japan"[5]. Although Upper Stillwater Dam was an important development in the progress of RCC for mass concrete, it was by no means the only recent construction that took place.

From 1984 to 1986, a number of small to moderate height RCC dams were constructed in the United States, using a variety of technologies for seepage control, lift bonding, and construction. These include Winchester Dam, Middle Fork Dam, Galesville Dam, Monksville Dam, Grindstone Canyon Dam, Stagecoach Dam and others. During this same time, RCC was used very successfully on several projects for rehabilitation of existing embankment and concrete dams and spillways.

RCC construction by USACE of Elk Creek Dam, Oregon, began in 1986 and continued in 1987 until construction was interrupted for environmental reasons. This 249-foot high dam is considered the current model for RCC design and construction by USACE.

Current RCC Dams Under Design or Construction

"As with conventional concrete, there does not appear to be a limit to the size of structure that can be designed and built with RCC"[5]. Some of the recent RCC high dam projects include:

High RCC Dams	Height (ft)	Status
Tamagawa, Japan	338	Highest completed to date
Bakaigawa, Japan	377	Under construction
Gatssan, Japan	410	Under construction
Miyagase, Japan	509	Under construction
Jamrani, India	492	Feasibility study completed
Kwan-in-temple, China	344	Feasibility study completed

CONCRETE DAM SECTIONS AND APPURTENANT STRUCTURES

General Dam Sections and Features

RCC dams can be built to a variety of cross sections, using several different face slopes. Outlet works, penstocks, spillways, stilling basins, and control structures can all be incorporated in the RCC design. Following is a general discussion of how these features can be designed and constructed.

Upstream and Downstream Slopes. - The upstream face of RCC dams is most commonly vertical. To construct such a face it must be formed. The upstream face should be designed to be as total a seepage barrier as possible. For this reason, recent designs without slipformed faces have called for a 3-foot wide section of conventional concrete, placed against the form and integrally vibrated with the RCC. Downstream face slopes can vary from 0.6-1.0 horizontal to 1.0 vertical. If left unformed, most RCC mixes will fall to slopes between 0.8 and 1.0H to 1.0V. Steeper face slopes must be formed. This can be done using the same methods as used for the upstream face, except seepage control is not critical on the downstream face.

Outlet Works. - The strategy in designing conduits and penstocks in an RCC dam is to try to minimize interference with the concrete construction. The conduits themselves are usually made of conventional cast-in-place or precast concrete and are often constructed in a trench cut into the foundation rock. A control tower can be built on the upstream face and use the dam for support. Stilling basins are usually constructed with RCC floor slabs and conventional concrete walls.

Galleries. - Galleries can be built into an RCC section to serve as access to the interior for inspections, a seepage collecter, access for instrumentation, and a terminal point for drain holes drilled into the foundation. Galleries and their locations within the dam section must be considered during a stability analysis. Gallery construction does decrease the efficiency of RCC construction. Past experience has shown RCC productivity decreases about 15 percent while constructing around a gallery.

Spillway. - For RCC dams, the spillway is incorporated into the structure, eliminating the need for a side channel design. Spillways can be designed for a variety of capacities, but an added feature of RCC dams is that they can be designed to overtop since RCC provides erosion resistance. Spillway designs for past RCC dams have generally fallen into two categories. The first is the traditional, smooth-surfaced, ogee profile. The second relies on the development of a protective turbulent boundary layer at the base of the flow. This second design has been constructed by building stair steps out of slipformed concrete elements. This design can eliminate the need for large baffle blocks. However, this design may be inappropriate for large spillway flows.

CONCRETE MATERIALS AVAILABILITY

Required Quantities

The amount of concrete and the amount of materials to construct an RCC dam will vary depending upon the site that is chosen and the final selected size and design. To formulate a preliminary estimate of material quantities required the following assumptions were made: (1) a vertical upstream dam face; (2) a slope of 0.75H to 1.0V on the downstream face; (3) a dam height of 420 feet; and (4) mixture proportions similar to those used for Elk Creek Dam, Oregon. Final quantities required will depend on the plan finally selected and the M-CACES design. Approximate material quantities required for comparison to available quantities for flood control structures at the 20.1 site and the 19.0 site are listed below.

SITE		20.1	19.0
CONCRETE	(cu.yds)	2,100,000	1,500,000
CEMENT	(tons)	130,000	90,000
POZZOLAN	(tons)	60,000	50,000
COARSE AGG	(cu.yds)	1,120,000	800,000
SAND	(cu.yds)	610,000	440,000

NOTE: These quantities are approximate and used only to evaluate adequacy of investigated aggregate sources. They do not reflect quantities which will be developed for alternative dam sizes or final selected plan.

Aggregate Sources

The large amount of aggregate required for this project necessitates the development of aggregate sources with close proximity to the project site, to minimize transportation costs.

Locations. - Alluvial aggregate along the Middle Fork of the American River upstream of the 20.1 site is the closest aggregate to the project site and contains both coarse and fine aggregate. The aggregates are predominantly located in bars along the river between Murderers Bar and Cherokee Bar, as shown in Plate 1. Cherokee Bar is the farthest from the project site. The distance from Cherokee Bar to the 20.1 site is approximately 15 miles, along the river. The distance to the 19.0 site is an additional mile. The river meanders in the canyon with the aggregate bars located on both sides of the river.

Quantities. - The USBR conducted explorations of the aggregate bars as part of their work on Auburn Dam in the late 1960's and 1970's.[7,8] These explorations were conducted to determine depths and volumes of usable aggregate in each of the bars. The bars varied widely in volume and depth, with depths from 10 to 80 feet. The total amount of aggregate in the bars evaluated was approximately 6.9 million cubic yards, with another approximately 1.0 million cubic yards of aggregate in the stream channel, for a total of 7.9 million cubic yards. The proportion of sand in these bars was approximately 31 percent of the pit run material. The usable portion of sand would

total about 20 percent of the pit run. Additional fine aggregate can be manufactured through the crushing of coarse aggregate. A cubic yard of aggregate would yield approximately one cubic yard of concrete.

Quality. - Testing by the USBR found the aggregate from the bars to be reasonably well graded in its native state. The maximum size of the aggregate was approximately 6 inches, with some oversize material. The aggregate is hard and sound. The sand had an average specific gravity of 2.59, and the absorption averaged 3.3 percent. The quality of the sand was determined to be acceptable, but was identified in the Bureau of Reclamation's aggregate investigation as "not the most desirable" due to the absorption and silt coatings on the sand particles. The coarse aggregate appears to be high quality material.

Production. - Production of aggregate would include washing, crushing and screening of the aggregate. Washing and screening of the fine aggregate would also be necessary, including the possible use of a scrubber to remove silt coatings on the sand particles. The aggregate production plant location choices are limited due to the steep terrain. Due to large amounts of water needed for aggregate processing, a site near the river would be desirable, as long as it was above flood elevation.

Dam Expandability Implications. - The quantity of aggregate available in the bars along the river exceeds the required amount for flood control dam alternatives being evaluated. The amount of aggregate required by a possible future expanded dam is expected to be less than the volume of aggregate in the bars. However, additional aggregates will be needed to meet the sand requirement and other deficient sizes. Additional aggregate for the construction of the expanded dam may be obtained from debris of the failed cofferdam at the 20.1 site and from an amphibolite outcrop on the north side of the river near Mammoth Bar. The flood pool created by the flood control structure would inundate all the bars upstream for short periods of time. However, since this water would be stored for a maximum of three weeks, the upstream bars would be accessible for use in building an expanded dam. Should a future expanded dam unexpectedly require more aggregate than is in the bars, additional aggregate could be processed from a quarry which could be developed in the area.

Cementitious Materials

Portland cement and pozzolan will be needed in large amounts to satisfy the requirements of this project. These materials can be transported in bulk by either rail or truck to the project site. Transportation by rail would be preferable due to the large amount of material required, and because storage in rail cars would be more efficient.

Cement. - There are four cement producers in the vicinity of the Auburn Dam project site. These producers are Genstar, Kaiser Cement, Nevada Cement, and RMC - Lone Star. All of these producers are located within 200 miles of the project site and produce Type II cements that have been prequalified for USACE use by the Waterways Experiment Station.

__Pozzolan__. - Pozzolan is available from two producers locally. Western Ash markets pozzolans from a terminal in Stockton, California, and Pozzolanic International markets pozzolans from a terminal in Sacramento. Both Class F and Class C pozzolans are available at these terminals. Each producer markets pozzolans from several sources at the terminals. The use of natural pozzolan sources within a reasonable distance to the project site may prove desirable. Two sources in the area include one near Hallelujah Junction, which is inactive, and one in Nevada, which is being used by the Nevada Cement Company. An undeveloped source of natural pozzolan near Grass Valley, California, may also be useful.

Water and Admixtures

__Water__. - A large amount of water will be required on this project, not only in concrete production, but also for curing of the concrete and for general construction. The water used in concrete mixes must be free from injurious materials. Water from the American River should be suitable for the purpose of concrete construction and of adequate supply.

__Admixtures__. - Water reducing and set retarding admixtures have been used in RCC mixes and may be beneficial, since the exposed surface would be large and the climate can be hot and dry. These admixtures, along with air entraining agents and high-range water reducers, may also be used in either bedding mixes or in the facing concrete.

CONCRETE PROPERTIES

General Concrete Properties

The following is a general discussion of concrete properties pertinent to a gravity dam. The properties are discussed as they relate to RCC.

__Compressive Strength__. - This is a measure of a concrete's resistance to crushing. It is the highest strength concrete exhibits and is rarely critical in the stability of a gravity structure. In RCC, the compressive strength is largely dependent on the paste volume in the mix. RCC is a very dry material, and some mixes may not contain enough paste to fill all the voids between aggregate particles. These entrapped air voids will reduce compressive strength. The latest RCC mixes have been designed to have a large paste volume, and high paste to mortar ratio, achieved primarily by using high percentages of pozzolan. These mixes were designed primarily to increase density, decrease permeability, and increase tensile and bond strength, but they were also able to achieve high compressive strengths.

__Tensile Strength and Strain Capacity__. - Tensile strength is about 5 to 15 percent of the compressive strength. The tensile strength is directly related to the strain capacity, which is the amount of strain a concrete section can endure before cracking. Tensile strain in a gravity section is most commonly caused by thermal contraction, but it can also be caused by seismic loadings. A concrete's ability to

withstand seismic or thermal strains without cracking stems from its tensile strength. The strength has been found to increase if crushed aggregates are used in the mix instead of rounded aggregates, or if a high paste volume is used in the mix. In addition, the dynamic tensile strength will always be somewhat higher than the static tensile strength.

Shear and Bond Strength. - Since RCC structures consist of a multitude of thin lifts, the bond strength between lifts is very important. If tensile strength is required across a lift joint, a high-slump bedding mix is usually used to glue the lifts together. Alternatively, the mix can be designed to have a high paste volume which will provide the necessary bonding. Cores taken from recently constructed RCC dams have established that such treatment develops a tensile bond strength at least as great as the tensile strength of the concrete itself. The bedding mix also greatly reduces seepage along lift joints. The shear strength along such a bonded joint will be a combination of cohesion from the bedding material and friction along the lift surface.

Elastic Properties. - These include Poisson's ratio, elastic modulus, and creep. Poisson's ratio has not been studied in great depth for RCC but those tests that have been done show the ratio for RCC is the same as for conventional concrete. The elastic modulus is very dependent on the aggregate in the mix. In general, the lower the aggregate quality, the lower the elastic modulus. Tensile strain capacity will increase as the elastic modulus decreases. Creep is deformation due to a sustained load over time and is a property of the paste in the mix. High creep indicates high stain capacity and increased resistance to thermal cracking.

Thermal Properties. - The thermal properties of RCC depend on the amount of cement in the mix and the type of aggregate used. Since cement hydration is an exothermic reaction, the amount of heat generated within a concrete mass will increase as the cement content increases. The amount of strain this heat generation imparts on the concrete depends on the coefficient of thermal expansion which in turn depends primarily on the aggregate used. Since RCC mixes usually contain less cement, they experience a smaller temperature increase than conventional mixes. Also, using large percentages of pozzolan to increase paste volume, instead of cement, reduces the heat gain. Lastly, the RCC method of placing thin lifts from abutment to abutment allows heat to dissipate more rapidly than conventional mass concrete block construction.

RCC DAM CONSTRUCTION CONSIDERATIONS

Diversion of Water

Mile 20.1 Site. - One positive aspect of the 20.1 site is that prior construction by the USBR has left a diversion tunnel that diverts the water in the river around the construction area. This is beneficial in the time and monetary savings to construct this anew. A cofferdam must be constructed at this site to replace the previous one that failed in 1986. Some of the original cofferdam remains in place

and could be utilized, reducing transportation and construction costs. The size of a cofferdam required for an RCC project is smaller than the original cofferdam that was constructed. This is because overtopping of the RCC structure during construction would not harm the concrete in place.

Mile 19.0 Site. - The 19.0 site has not had any previous construction for the diversion of the water around the site. At this site, a small cofferdam would need to be constructed before diversion of water would be possible. One method of diverting water would be to construct a conduit that would pass through the dam. Once the water has been diverted through the conduit, the remainder of foundation work and dam construction could commence. Another possibility would be to divert water in a similar manner as was done at the 20.1 site. Because the gross pool level of Folsom Lake is higher than portions of the 19.0 site, a downstream cofferdam may also be necessary.

Foundation Treatment

Mile 20.1 Site. - Foundation work at the 20.1 site was performed by the USBR, which included clearing of the site area, foundation and spillway excavation, and dental concrete placement in the foundation. Some of this work could be utilized for future dam construction, including a portion of the dental concrete. Some additional foundation treatment would be necessary due to the new dam configuration and alignment that has been proposed. A report has been prepared by Geology Section on necessary exploration and site studies required at both the 20.1 and the 19.0 sites.

Mile 19.0 Site. - More foundation work will be necessary at the 19.0 site than at the 20.1 site. Extensive explorations and foundation studies will need to be conducted at the 19.0 site to determine site conditions. Foundation preparation would include clearing and excavation followed by dental and levelling concrete work.

Access

Mile 20.1 Site. - Access to the 20.1 site has been developed as part of earlier construction. Some of the access has been obstructed due to past flooding. Additional access will be needed for aggregate transportation and for concrete plant operations.

Mile 19.0 Site. - Access to the 19.0 site would have to be developed extensively. Roads will be needed for construction access to the river as well as both abutments. Access will also be needed here for aggregate transportation, aggregate processing operations and for the concrete plant(s).

Concrete Production

The construction of a roller compacted concrete dam will require the production of roller compacted concrete and conventional portland cement concrete. A concrete plant will be needed for the production of each of these types of concrete. Production rates will be dependent on material delivery and plant capacities.

Aggregate Delivery. - Aggregate excavation can be done with bulldozers and front end loaders down to river level, and with draglines below water level. Aggregate must then be transported from the excavation site to the processing plant and after processing to the concrete plant. Aggregate excavated from bars along the river can be transported to the processing plant by various methods. A conveyor system can be constructed along the river to carry the aggregates, and provide a continuous delivery of aggregate to the processing site. Another option for transporting aggregates would be the construction of a rail system along the river. An old rail grade that exists along some reaches of the river could be utilized. Conveyors could be used between the rail system and the processing plant. Trucks are another possible method for transporting the aggregates. However, trucking aggregates would probably not be done because of the large volume of aggregates and the length of the haul. One benefit of using a rail system would be the ability to store aggregate in rail cars. This is important, since area for aggregate storage is limited. After processing, the mode of transportation of the aggregate to the concrete plant would depend on the distance from the processing plant to the concrete plant. Locating the concrete plant(s) adjacent to the processing plant would allow the use of short conveyors between plants. However, if topography does not allow the aggregate and concrete plants to be adjacent, either rail or conveyor systems similar to those used to transport aggregate to the processing plant could be used.

Mix Plant Location. - The concrete plant(s) will require areas for operation at a close proximity to the construction site. Ideally, the RCC concrete plant should be located as close to the aggregate processing plant as possible. Doing so minimizes handling of crushed and graded aggregates and reduces breakdown and waste. The aforementioned conditions greatly restrict potential plant locations. As conventional concrete requirements will be considerably less than that for RCC, the batching and mixing plant for conventional concrete may be located away from the aggregate processing plant. One potential plant location at the 19.0 site is where a plant was located previously, on the right abutment. At the 20.1 site, locations on both abutments may be useful, although the right abutment would be preferable because of easier cement and pozzolan delivery. Locating concrete plants on the abutments would also be suitable for the delivery of concrete to the placement site. Conveyor systems are considered most desirable for RCC delivery, though trucks have been used.

Mixer Type. - RCC may be mixed in the same type of drum mixer as conventional portland cement concrete, but mixing in a pugmill type mixer is more desirable and practical. Multiple mixers will be necessary to accommodate the high placement rate that is capable of being achieved in RCC construction. A separate drum mixing plant will be required for mixing conventional concrete and bedding mortar.

RCC Construction Methods

RCC is portland cement concrete. Although it is identical to conventional concrete in many of its properties, it differs, however, in its placement and the methods used in its construction.

<u>Foundation Treatment</u>. - The foundation treatment for an RCC dam involves the excavation and dental concrete work that would be required of a conventional concrete dam. In their feasibility design summary for a curved multi-purpose gravity dam, the USBR outlined a foundation treatment plan that included a grout curtain up to 280 feet deep and consolidation grouting 30 feet deep. Conventional concrete is necessary along the base of the dam as a levelling surface from which to begin the RCC placement. This levelling concrete provides a base and clean surface on which to place the RCC. Applying RCC to a clean surface is essential to allow the development of adequate bond and shear strength.

<u>Lift Construction</u>. - RCC is placed in layers or lifts that extend the full width and the full length of the dam. After a lift is completed, another lift is placed upon the previous lift. Lifts are typically placed in thicknesses of up to two feet. Concrete for each lift is spread using bulldozers or similar equipment and then compacted with vibratory rollers. To avoid segregation, lifts are placed in multiple thinner layers with the tracks of the bulldozers providing initial compaction. Bedding concretes and mortars have been used between lifts to increase bond and shear strength. Time between lifts and ambient temperature play an important role in achieving the desired bond strength between lifts. Water is needed for curing and to maintain freshness of lift joints. Lift surfaces should be kept moist, not wet, and lifts should be sloped to prevent pooling and allow water to drain from the lift surface. Cleanliness of lift surfaces is also essential in the development of bond strength between lifts. High pressure water hoses can serve the dual purpose of cleaning the lift surface and keeping moisture on the surface.

<u>Joints</u>. - Contraction joints have been installed in RCC dams using galvanized steel sheeting vibrated into the RCC with a vibratory inserting device mounted on a backhoe. Waterstops have also been used in RCC dams. Waterstops have been installed at the upstream facing elements along with vertical drains. The drains are positioned between each double waterstop. The lift joints, as discussed above, may be constructed either with or without joint treatment.

<u>Upstream and Downstream Faces</u>. - Construction of RCC dam faces has been accomplished in various ways. Slipform pavers, precast concrete panels, and jump forms have been used to construct both the upstream and downstream faces of RCC dams. In conjunction with the use of jump forms or precast concrete panels, a conventional concrete section one to three feet wide is placed to provide a water barrier. Slipformed faces, however, do not require the additional concrete barrier and have the advantage of compaction of RCC up to the face, which provides bond. RCC has the advantage of allowing the downstream face to be placed without forms. In constructing the downstream face in this manner, the downstream face is overbuilt and the additional concrete is sacrificial.

<u>Spillway</u>. - An RCC dam can have a spillway incorporated into the dam itself. An ogee design can be constructed using a shotcrete overlay or by anchoring a layer of formed conventional concrete to the

previously placed RCC. The other option is to place the RCC lifts along the spillway in stepped fashion, using either slipform facing or jump forms. The construction of a stepped spillway reduces the spillway cost because it reduces construction time. Whichever design is used, conventional, high-durability concrete should be used for the spillway surface.

Galleries. - Galleries can be constructed using the slip form method, with precast facing elements, or with jump forms. The roof of the gallery may be constructed by placing a precast slab or culvert on top of gallery walls. This provides a gallery which requires no additional construction effort when completed. Another method which has been used is the replacement of RCC with fine aggregate in the gallery areas. Once constructed, the aggregate is excavated from within the galleries.

Rate of Construction. - The rate of construction of RCC dams typically is much greater than the rate for conventional concrete dams. The rate of RCC placement will vary depending on the production capacity, lift depth, width of the dam section, discontinuities such as galleries, and equipment. Placement rate is usually slow at the base of the dam, but as the surface area of the lift increases allowing equipment to move about more easily, the placement rate increases. Near the top of the dam, as the width of the dam section narrows, placement rate once again decreases. At the Corps of Engineers' Elk Creek Dam project, peak production rates were estimated at 15,000 cubic yards per day. Production exceeding 10,000 cubic yards per day was achieved early in construction with four pugmill mixers. Other dams, such as Upper Stillwater Dam in Utah, have attained production rates in excess of 10,000 cubic yards per day. An average rate of RCC production for Auburn Dam of 10,000 cubic yards per day is feasible using current technology.

Construction Costs. - Because of the use of efficient earthmoving equipment, and some savings in materials and labor costs, RCC for dams is very inexpensive. Recent RCC dam costs, including all costs associated with concrete, have ranged from $19 to $27 per cubic yard, for dams similar to a concrete dam at Auburn. $22 per cubic yard represents a reasonable estimate for the cost of RCC for any Auburn Dam site and structure.

CONSIDERATIONS AND EXPANDABILITY OF A CONCRETE DAM

A number of considerations enter into the design and analysis of concrete dams, and affect the expandability of such dams. Table M-6-1 summarizes advantages and disadvantages of an RCC dam at either the 20.1 or the 19.0 site. Significant items concerning a concrete gravity flood control structure and expandability of a concrete gravity dam follow.

TABLE M-6-1

ADVANTAGES AND DISADVANTAGES
OF A FLOOD CONTROL DAM
AT THE 20.1 AND 19.0 SITES

ADVANTAGES	DISADVANTAGES
20.1 Site	
Low cost	Design for 9" fault displacement
Rapid construction	Larger volume of RCC than 19.0 site
High degree of safety	
Bypasses F-1 fault trace	
Fdn treatment partly done	
No side channel spillway	
Easily expandable	
No spillway to fill in if expanded	
No new fdn exploration needed	
Plentiful supply of materials	
No environmental problem getting matls	
Simple, low cost diversion	
Overtopping protection built-in	
Early flood protection	
No D/S cofferdam	
Less fdn treatment for expanded dam	
Good access to site	
Rail spur avail.for cement/pozz deliv	
Shorter transport for aggregates	
19.0 Site	
Low cost	Design for 9" fault displacement
Rapid construction	Fault system unknown
Smaller vol of RCC than 20.1	Fdn exploration needed
High degree of safety	More fdn work needed than 20.1
No side channel spillway	More expan.dam fdn work than 20.1
Easily expandable	D/S cofferdam required
Early flood protection	Difficult site access
Simple, low cost diversion	Expan dam powerplant infeasible?
Overtopping protection built in	
Plentiful supply of materials	
No spillway to fill in if expanded	
No environmental problem getting matls	

Concrete Dam Safety Issues

The safety of concrete dams has been well documented. No concrete dams have failed due to an earthquake, although many have been subjected to moderate to severe shaking. No concrete dams have failed in the United States in the last 50 years, and earlier failures were due to unknown foundation conditions prior to the advent of foundation engineering. The few failures of concrete dams worldwide have been due to foundation problems, not the concrete structure. Concrete dams have successfully withstood numerous incidents of overtopping with minimal or no damage, whether due to floods or landslides into the reservoir. Concrete dams have been overtopped during construction, with minimal or no damage. Concrete dams are not subject to failure due to water seepage, or many other predominant causes of dam failure.

Fault displacement at the F-1 fault at the 20.1 site, or at any of the potential Auburn Dam sites, has been a concern. A straight concrete gravity dam at the 20.1 site appears to be downstream of the F-1 fault, rather than being built on the fault. This appears to increase the safety of this structure. Where active faults have been found within the foundations of concrete dams, moveable joints have been built in the dams to accommodate this potential movement. Morris Dam, a concrete gravity dam constructed in 1934 near Los Angeles, has a special transverse joint to accommodate foundation movement of up to 3-feet. USBR and their Board of Consultants for the Auburn Dam have addressed the issue of fault displacement in their design for the CG-3 structure at the 20.1 site. ICOLD Bulletin 52, Earthquake Analysis Procedures For Dams, discusses this concern, and includes the following statement:

"In particular the possibility of a movement on a fault zone should at all times be avoided by a suitable geological study. Often, it is speculated that in the event of such a movement, 'soft' structures - e.g. earth dams - are safer than more rigid concrete ones. This prediction is however beyond the realm of calculations which are feasible at present, and it certainly is possible that a concrete gravity dam is safer than an earth dam in such a fault movement due to its inherent stability even after damage."[9]

Dreher[10] provides an outstanding analysis of seismic design considerations for concrete dams, including design for fault displacement in a dam foundation. An additional consideration for a flood control structure is that the occurrence of fault displacement at a time of full reservoir pool is very unlikely.

Expandability of RCC Dams

Several general items to consider when addressing dam expandability are discussed below.

(1) If an embankment dam were built at any Auburn site, expansion of the dam with concrete would be virtually impossible. The embankment dam could be used as a cofferdam for a new concrete dam downstream of the cofferdam site, but this would be very unlikely.

(2) Expansion of an RCC dam to a larger embankment section may be feasible, using the RCC as part of an impervious zone. Design of this type of structure would be untypical of embankment dams, but may be practical.

(3) Expansion of an RCC dam to a larger section of RCC would be relatively easy for any Auburn site. Bonding of an RCC section to an older RCC section would be relatively easy and practical. One method to provide a surface for future bonding would involve constructing the downstream face of the lower dam with well compacted or formed steps. For expansion of the dam, these steps would be scarified, cleaned, and then treated with a bonding mortar before RCC placement. This would be the same bonding mortar used for bonding the lifts of RCC. A similar method was actually used for expansion of O'Shaughnessy Dam in Yosemite National Park, CA., in 1938, increasing the height of the dam from 345 feet to 425 feet, using slumpable concrete. Middle Fork Dam was constructed in 1984 with RCC using cast downstream face steps, and Knellpoort Dam in South Africa, completed in 1988, had a stepped downstream face on this RCC arch dam. The strength of the older RCC section would be used in the analysis of the new dam, and would have some affect on the downstream face slope design.

(4) Expansion of an RCC dam to a higher RCC dam would minimize foundation preparation, since only the portion of the foundation downstream of the older dam would have to be prepared.

(5) Any embankment dam at Auburn, if expanded, would require filling and raising of a remote spillway. This would not be necessary with an RCC dam.

FUTURE EFFORT

After a plan is selected, detailed design will begin on that plan. This design must take place before final plans and specifications are developed for the selected plan. This effort will take several years and involve many different professionals. Following are some of the design efforts to be done during this detailed design phase.

Future Materials Studies

Aggregate Studies. - Although plentiful coarse aggregates are available from bars on the Middle Fork of the American River, there may be a shortage of sand. Some additional investigation of natural or crushed sand sources may be appropriate to optimize aggregates for mix proportioning. This would involve some field work, sampling and testing of aggregates, prior to mix proportioning. Feasible measures to preserve as much natural aggregate as possible for a future expanded dam may be advisable.

RCC and PCC Mix Proportioning Studies. - Although USBR has conducted extensive mix proportioning studies for conventional concrete, none have been done for RCC. RCC mix proportioning studies would be conducted to determine cementitious material contents,

aggregate proportions, and various concrete properties for use in final design. Testing may be conducted at NPDL, WES, or another USACE laboratory.

<u>RCC Properties</u>. - Testing for RCC properties is usually considered part of the mix proportioning process. However, some properties, such as thermal properties and rapid strength, may be tested for at a laboratory other than that doing the mix proportioning. Thermal and creep property testing may be performed at NPD Laboratory, and the rapid strength testing would probably be performed at UC-Berkeley. WES may do some of the testing for RCC as well.

Concrete Dam Design

Design of concrete gravity dams in USACE is normally accomplished with a team of materials and structural engineers. Design of a concrete dam at Auburn would likely follow this path, since the concrete properties and structural design are interdependent. Concrete materials engineers should participate in the structure design process at all stages. Feasibility level structural design has already begun. Final structural design usually begins after the completion of concrete property testing. Static design will probably involve a two dimensional analysis, and dynamic analysis will likely involve a three dimensional analysis using a finite element method (FEM) approach. Analysis of thermally induced stresses can be done satisfactorily using two dimensional FEM.

CONCLUSIONS

a. Any concrete gravity dam built at Auburn should be built with RCC. RCC properties are more than acceptable for any size or height of dam at Auburn.

b. Concrete aggregate supplies are more than adequate for a flood control dam at any Auburn damsite. These aggregates have been fully studied and documented.

c. Concrete aggregate supplies appear to be adequate for a multi-purpose dam at any Auburn damsite. These aggregates have been fully studied and documented. Careful planning is needed to assure that the alluvial aggregates along the Middle Fork of the American River are available for an expanded dam.

d. Using streambed aggregates for the concrete gravity dam eliminates the environmental problem associated with stripping borrow materials from outside the reservoir area.

e. Plentiful cement and pozzolan supplies are available for construction of any size dam at Auburn.

f. The ability of a concrete dam to survive a seismic event with a displacement of 9 inches is an important concern for design, but should not be overemphasized. The inherent safety of concrete dams in such an event have been supported by many distinguished authorities, including ICOLD. Current USBR Auburn dam designs consider fault displacement.

Bechtel National, Inc., and the USBR's Board of Consultants (Wallace L. Chadwick, William R. Gianelli, Ralph B. Peck, and Ernest K. Schrader), have reviewed the current design. A concrete gravity dam can be safely designed and built at the Auburn Dam site.

g. Design for fault displacement will be a part of any dam design for any Auburn damsite.

h. The 19.0 site has several problems with expandability that should be resolved before this site is considered for a dam.

i. Although any gravity dam at the 20.1 site has a slightly larger volume of concrete than the 19.0 site, a dam at the 20.1 site has several advantages with expandability over the 19.0 site.

j. Structural design engineers will determine the recommended concrete dam footprint and section.

k. A flood control concrete dam should be designed as a stand-alone structure, as well as part of an expanded dam.

l. RCC dams can be built in a substantially shorter time than either a conventionally placed concrete dam or an embankment or rockfill dam of the same reservoir capacity. At RCC placement rates that have already been attained at other RCC dams, the RCC for a flood control dam at any Auburn site could be placed in as short a time as one (1) year.

m. Because of the rapid rate of RCC placement, a concrete dam at the Auburn site would provide flood protection at an earlier date than an embankment or rockfill dam.

n. Feasibility level design of a flood control gravity dam should be closely coordinated with USBR design for a multi-purpose Auburn Dam.

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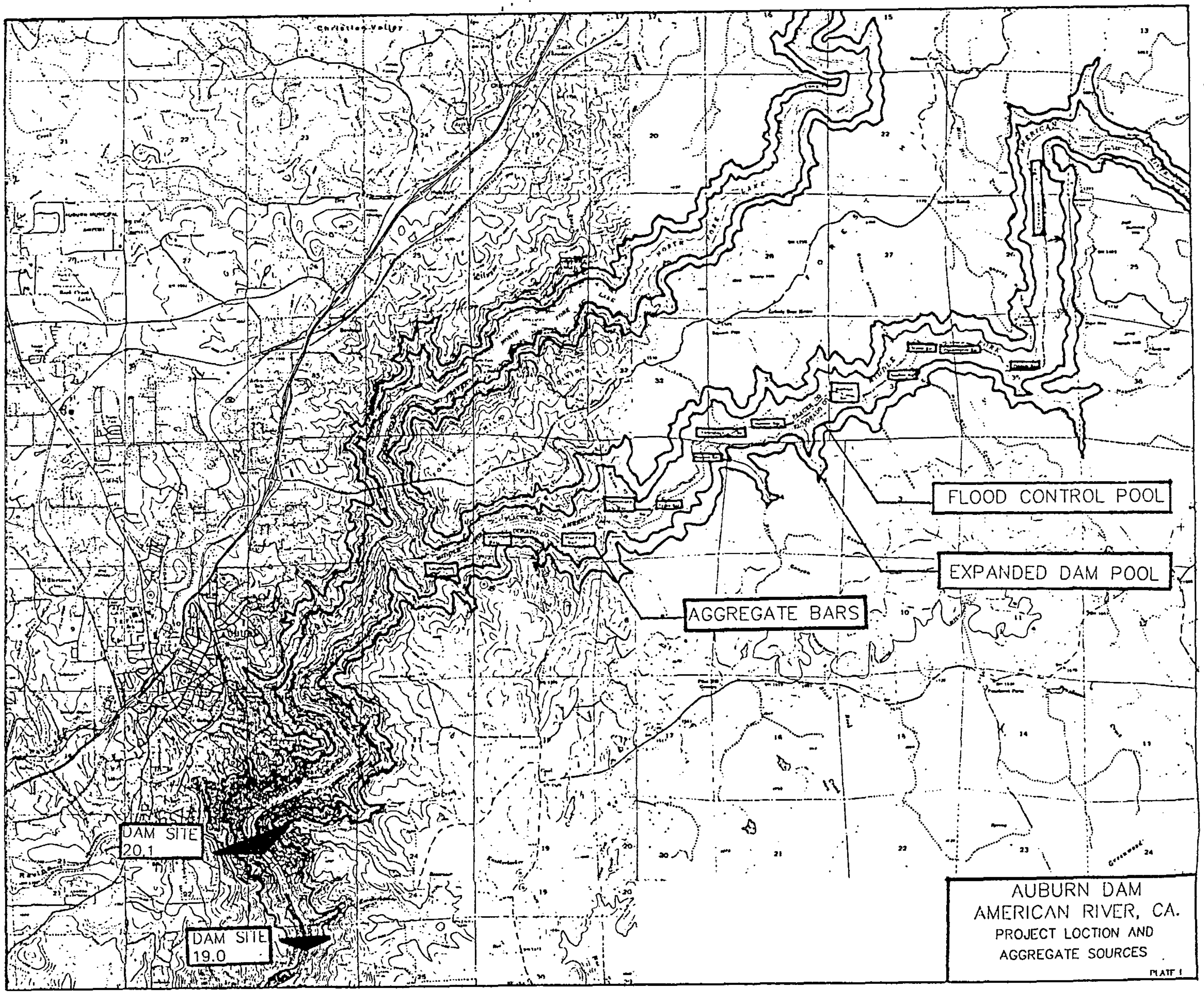
FLOOD CONTROL POOL
EXPANDED DAM POOL
AGGREGATE BARS
DAM SITE
20.1
DAM SITE
19.0
AUBURN DAM
AMERICAN RIVER, CA.
PROJECT LOCTION AND
AGGREGATE SOURCES
PLATE 1

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 7

RESERVOIR RIM
AND
SLOPE STABILITY STUDY

JUNE 1990

PREPARED BY

GEOLOGY SECTION
GEOTECHNICAL BRANCH

AUBURN DAM AND RESERVOIR
RESERVOIR RIM AND SLOPE STABILITY STUDY

TABLE OF CONTENTS

PHOTOGRAPHS OF SELECTED SLOPE STABILITY PROBLEMS
NORTH AND MIDDLE FORKS, AMERICAN RIVER

NOTE: PHOTOGRAPHS NOT INCLUDED IN THE DOCUMENTATION REPORT ON FILE WITH GEOLOGY SECTION, GEOTECHNICAL BRANCH, SACRAMENTO DISTRICT, CORPS OF ENGINEERS

TABLE OF CONTENTS (CONT.)

PLATES

NOTE: PLATES NOT INCLUDED IN THE DOCUMENTATION REPORT ON FILE WITH GEOLOGY SECTION, GEOTECHNICAL BRANCH, SACRAMENTO DISTRICT, CORPS OF ENGINEERS

AUBURN DAM AND RESERVOIR
RESERVOIR RIM AND SLOPE STABILITY STUDY

INTRODUCTION

The Geology Section conducted a reservoir rim and slope stability study of the area upstream of the River Mile 20.1 damsite between the river and approximately elevation 900 feet. The purpose of this investigation was to locate and identify landslides and areas of potential slope instability in the reservoir area to be formed by the proposed flood control dam at Auburn, California. The evaluation was based strictly on the geologic and topographic conditions evident from aerial reconnaissance by helicopter and limited ground-truthing, combined with work previously conducted by the U.S. Bureau of Reclamation (USBR). Due to the complex geologic nature of the study area, a complete and detailed study would have required an extensive and cost prohibitive surface investigation and subsurface exploration program. In this report the term "slide" is used as a generic term to denote any slope failure, without regard to its structural form.

SCOPE OF WORK

The initial phase of the study was to obtain the existing black and white, and color aerial photographs. Photographs (scale 1"=800') taken in October 1986 following the failure of the USBR Auburn Dam cofferdam were studied, and all suspected slides were located and plotted on USGS 7.5 minute series topographic quadrangle maps. Areas of ancient slides and suspected slope instability as located by the USBR during their studies in the Robie Point area, and in the vicinity of their proposed Greenwood Bridge on the Middle Fork of the American River, were also plotted on the topographic maps.

A major part of the study consisted of conducting a series of helicopter flights over the proposed reservoir area to observe and locate on maps the existing slides in areas too remote for reasonable ground access. Both sides of the canyon walls of the North and Middle Forks of the American River were video taped during the helicopter flights. Audio narration was included as an aid to locating specific features when viewing the tapes. The slides observed during the helicopter flight and recorded on video tape were then plotted on the topographic maps. Emphasis was placed on video taping of the slides caused by the rapid lowering of the reservoir following the cofferdam failure in 1986. It was felt that the cofferdam failure represented the worst case scenario for rapid reservoir drawdown on the stability of the canyon walls. The surface of the reservoir at the time of the cofferdam failure was approximately elevation 700 feet.

Following the areal reconnaissance, several trips were made to ground-truth the accessible areas along the river. In addition to the slides observed from the air, numerous roadcuts and rock outcrops were field checked for indications of slope instability.

As with the aerial survey, the major emphasis of the ground reconnaissance centered on the reservoir area below the 700-foot elevation. Additional slides and areas of concern which were observed on the ground were also delineated on the topographic maps. Photographs were taken of several of the slope failures and are included as Appendix A. The numbering of the slides on the topographic maps (Plates 1 through 4) corresponds to the photograph numbers in Appendix A.

GEOLOGY

The bedrock in the proposed dam and reservoir area consists of complexly folded and faulted metamorphosed sedimentary and volcanic rocks. The layering of the original sedimentary and volcanic rocks parallels the regional metamorphic structure which strikes northwesterly and has steep northeast to vertical dips. Locally, the structure is complicated by faulting and intrusions of granitic-dioritic and ultrabasic rocks. Most of the ultrabasic rocks have been completely altered to serpentine and talc. Faults and weak rock zones are common parallel to the major structural trend and occur in some places across the trend.

In their report on the Robie Point area, the USBR included the following descriptions and characteristics of the major rock types located in the area (USBR, 1966):

> Amphibolites are usually hard and schistose with prominent, steeply dipping, variably spaced cleavage planes parallel to the schistosity. These cleavage planes are a distinct weakness in the rock and vary from very closely spaced to widely spaced in more sound rock. The weak amphibolite zones usually contain some chlorite and talc schist (see description below). The amphibolites also contain small irregularly shaped intrusive bodies of hard, gneissic metagabbro. Weathering is variable, mostly affecting the weak rock zones.
>
> Metasediments are mainly thin-bedded slate with minor phyllite, quartzite, and chert beds up to 5 feet thick. The rocks are chiefly hard and generally part easily along the steeply dipping bedding planes. They are usually only slightly weathered and are fairly competent even near the ground surface.
>
> Serpentine is widely distributed throughout the area. It occurs as dikes and sills along fault zones and as large, discordant, irregular ridge-forming masses. It occurs predominantly as two types: one is massive and variably fractured; and the other is sheared, foliated, talcose, and usually unstable.
>
> Talc and talc schists are common along the sheared serpentine borders and within faults and weak rock zones. They are also associated with shear zones within the serpentine, metasediments, and with chlorite schist in the weak amphibolite zones. The talcs are very soft and slippery, and are the underlying cause of most of the landslides in the Robie Point area.

Overlying the bedrock along most of the canyon walls is a thin mantle of fragmental and unconsolidated rock material called regolith. The regolith is typically highly varied in character and is either formed in-place (residual) or is transported (colluvium). It includes rock debris of all kinds, as well as soil.

LANDSLIDES CAUSED BY FLUCTUATING RESERVOIR LEVELS

Landslide masses are typically triggered by natural processes that represent a complex interaction of material properties, geometry, and environmental conditions. Factors that most commonly exert an influence on the slope profile include the rock type, its inherent strength, the presence of joints within the rock, the geologic structure, the environment, the hydrologic conditions, and the tectonic and geomorphic setting (Rogers, 1989).

Slope failures can be divided into two groups, hard rock or bedrock failures, and soft rock or soil failures. The latter encompass failures involving the regolith that commonly lies upon the bedrock. Because the slope stability problems in the study area seem to occur primarily in regolith, the following discussion will be limited to that of soft rock or soil failures. For a complete description of all types of landslides, see Landslides: Analysis and Control (Schuster and Krizek, 1978).

The stability of the soft rock or soil slopes is controlled by the deterioration of unit strengths due to partial or complete saturation. Studies conducted for several large landslides associated with fluctuating reservoir levels suggest that a rise in the water table will cause a decrease in the effective stresses which aid in the inter-granular friction between soil particles within the regolith. If the regolith contains clayey material, often what occurs is the clay fraction swells, the soil mass becomes less permeable, and the total unit weight increases while the effective weight of the mass is buoyed by its submergence. In the event that the reservoir is rapidly lowered and the solid mass doesn't have sufficient time to drain the water, what occurs is an overall increase in the forces promoting failure concurrent with a decrease in friction at the soil/bedrock contact.

It should be noted that the reservoir behind the cofferdam that failed was released in a matter of a few hours. In comparison, the controlled releases under normal operation of the flood control dam with a reservoir at the same elevation (approximately elevation 710 feet) is estimated to take in excess of 3 days. The slower drawdown of the reservoir should allow more time for the water to drain from the slopes, and thus reduce the chances of slope failure.

LANDSLIDES IN THE RESERVOIR AREA

In the dam and reservoir area three major types of slope movement were identified; slides, debris avalanches, and topples. The largest number of these appear to be relatively small debris slides and debris avalanches which occurred in 1986 below elevation 700 feet as a result of the rapid lowering of the reservoir behind

the cofferdam following its failure. In this reconnaissance survey, at least 35 new slides or avalanches were found in the area below elevation 700 feet and upstream of the proposed dam alignment. Of these, about five appear to have been old features identified in previous studies which were rejuvenated in 1986. It appears that the lower elevations also correspond to the areas of the greatest amount of regolith accumulation.

Debris slides and debris avalanches differ in that avalanches move more rapidly due to lower cohesion or higher water content and steeper slopes. In debris slides, the moving mass breaks up into smaller and smaller parts as it advances toward the foot, and the movement is usually slow. In debris avalanches, progressive failure is more rapid, and the whole mass, either because it is so wet, or because it is on a steep slope, liquefies, flows and tumbles downward, and may advance well beyond the foot of the slope. Debris avalanches are generally long and narrow and often leave a serrate or V-shapped scar tapering uphill at the head (Schuster and Krizek, 1978).

The largest slides which have been identified in the proposed reservoir area are two ancient rotational or translational slides. One is located on the northwest side of the river at River Mile 22.4 (see Auburn Quadrangle map, Plate 1) approximately 1-mile downstream of the confluence of the North and Middle Forks of the river, and the other is at Cherokee Flat (see Greenwood Quadrangle map, Plate 2). They were identified by the USBR during investigations for an alternative damsite at River Mile 22.1, and studies for the Greenwood Bridge and highway relocation proposal.

The River Mile 22.4 slide appears to be the remnant of an old stabilized landslide. The USBR examined road and railroad cuts and determined that the slide extends approximately 2200 feet upslope from the river and is 900 feet across at its widest point. Explorations for the alternative damsite indicate that the slide is a maximum of 200 feet thick. In 1986, a fairly shallow portion of toe of the slide fell into the river and was carried away. In the event that complete failure of the slope should occur, the river channel could be blocked. For this reason, a program should be initiated to periodically monitor movement of the slide should a dam be built downstream.

The USBR has identified about 15 fairly large old slides in the Greenwood Bridge area between Poverty Bar and Oregon Bar on the Middle Fork. The largest of these slides has the remnant of what appears to be a nearly flat and well developed slump block which is locally called Cherokee Flat. The slide seems to have originated approximately 1200 feet upslope of the flat as indicated by a roughly arc-shaped topographic expression. The total length of the body of the landslide is approximately 2000 feet from the river level to the top of the slump block. The elevation along this stretch of the river is approximately 680 feet, so water backed up by the cofferdam was fairly shallow. As a result, very little post-1986 slope failure is evident. Due to its large size and its proximity to the river, this slide should also be periodically

monitored for movement.

Approximately 2 miles upstream of Cherokee Flat, a translational debris slide occurred during the unusually rainy winter of 1939-1940. The slide occurred on the left abutment of the Ruck-A-Chucky debris control dam which was being constructed by the Army Corps of Engineers (but not completed). During excavation of the left abutment, a slide comprised of overburden and large loose blocks of rock from the hillside above the damsite came down, covering the left abutment and filling the river channel. The slide occurred when the material was lubricated by the rain and when the supporting material on the slope below was removed by excavation. Large blocks of slide material are still evident in the channel at the upper end of Ruck-a-chuckey rapids.

The only topple identified in the dam and reservoir area is the large slide discovered during construction of the thin-arch dam at River Mile 20.1, and which the USBR had designated Slide 16. As discussed in the Feasibility Report, due to the slide's location, partial or complete removed of the slide will be required during foundation excavation for the proposed dam.

CONCLUSIONS

* Portions of the canyon walls upstream of the proposed dam have had several episodes of prehistoric slope failure.

* Numerous small slope failures, mostly in the form of debris avalanches, occurred as a result of the filling and/or sudden release of the reservoir behind the cofferdam during the 1986 cofferdam failure.

* The area of the slides constitutes only a very small fraction of the total area within the reservoir limits.

* The slope failures which occurred in 1986 represent the portions of the slopes which were the most inherently weak, and thus susceptible to natural slope failure. The failure of the cofferdam only hastened its occurrence.

* Repetitive filling and emptying of the reservoir behind the proposed dam will continue to remove those portions of the slopes which are already prone to failure. It is impossible to determine the frequency and extent of future slope failure.

* Most likely, each episode of filling and emptying should cause fewer failures as the unstable portions of the slopes are gradually removed, and eventually the canyon walls should stabilize.

* No field evidence was found to indicate that the proposed dam or any of its appurtenance are in any jeopardy of being damaged or of losing their function due to slope failures.

* If the proposed flood control dam is approved for construction, a program of slide monitoring should be initiated for the slide at River Mile 22.4 and the Cherokee Flat slide. These were the only slides observed which could obstruct the flow of the river if they were to totally fail. A thorough discussion of field instrumentation and surveying of slides is included in Landslides, Analysis and Control (Schuster and Krizek, 1978).

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AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 8

EVALUATION OF SOILS AND SOIL STABILITY FOR THE PROPOSED FLOOD CONTROL DAM AT AUBURN

CALIFORNIA DEPARTMENT OF WATER RESOURCES

State of California
The Resources Agency
DEPARTMENT OF WATER RESOURCES
Central District

EVALUATION OF SOILS AND SOIL STABILITY FOR THE PROPOSED FLOOD CONTROL DAM AT AUBURN

Memorandum Report

October 1991

State of California The Resources Agency

Memorandum

Date : September 30, 1991

To : Jim McDaniel
Deputy Director

Jerry Vayder, Chief
Central District

From : Department of Water Resources

Subject : Evaluation of Soils and Soil Stability for the Proposed Flood Control Dam at Auburn

The Central District has completed an evaluation of the potential for soil loss in the inundation zone at the proposed Auburn flood control dam. This study was initiated in response to several comments on the draft EIS for the proposed project, which expressed concern over the operation of a flood control dam on the North Fork American River. The concern expressed by the commentors is that the proposed operation would strip much of the soil cover from the reservoir area, resulting in a significant loss of important wildlife habitat.

Results of our studies indicate that 35 percent of the soils in the inundation zone are stable under any operation. About 15 percent of the soils in the detention dam's inundation zone may mobilize at the drawdown rates proposed for both the 200-year and 400-year flood control dam. This results in a potential impact to about 600 acres of habitat. The operation of the outlet works for either 200-year or 400-year dam can be modified so as to limit the potential to destabilize soils. This modification of the outlet works design should be performed during preliminary engineering design when detailed field and laboratory testing of the soils can be combined with more precise modeling of the outlet works.

SURNAME			
DWR 155 (REV. 8/83)	Jerry Dudley 9-30-91	Steve Yaeger 9-30-91	Vayder 9-30-91

CONTENTS

Figures

Tables

Plate

Organization

State of California
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The Resources Agency
Douglas P. Wheeler, Secretary for Resources

Department of Water Resources
DAVID N. KENNEDY, Director

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INTRODUCTION

During the record flows of February 1986, the flood contol facilities that protect much of the metropolitan Sacramento area were taxed to their limits. River stages encroached into levee freeboard at many locations. Folsom Dam, the primary flood control facility in the American River watershed, was nearly filled to capacity. After the flood, studies by the Federal Emergency Management Agency concluded that the level of flood protection in the Sacramento area was much less than previously thought and that the area is in jeopardy in the event of floodflows.

The U.S. Army Corps of Engineers has undertaken studies to evaluate several alternative means of improving the level of flood protection in the Sacramento area. One alternative is construction of a flood detention dam near Auburn.

As part of the environmental documentation process for the proposed flood control dam, the U.S. Fish and Wildlife Service prepared a draft *Habitat Evaluation Procedures* report for the project, dated February 1991. The report was critical of the impacts the proposed dam would have on habitat in the reservoir area. The report concluded that:

- Soil and slope slippage and erosion would be significant.
- Potential soil loss or movement and its effect on the vegetation and habitat of the American River canyon was a more critical issue than extinction of particular plant species resulting from periodic inundation.

These conclusions were based on examination of aerial photographs of portions of the American River Canyon before and after the Auburn cofferdam was breached during the flood of 1986. The section of canyon examined in the HEP report extends upstream from the cofferdam about 4 miles, which is about 1 mile upstream from the confluence of the North and Middle forks of the American River. A major assumption in the Fish and Wildlife Service study was that this section of canyon is geologically and pedologically representative of the entire 23 miles of canyon that would be inundated by a permanent flood control dam.

This report provides an assessment of the amount of soil erosion and slope slippage that can be expected as a result of periodic inundation by the proposed flood control dam at Auburn. The type of analysis presented in this report was chosen so as not to rely on effects of the 1986 cofferdam breach, because such conditions are not necessarily representative of conditions that can be expected during normal operation of a permanent flood control dam.

SOILS IN THE AREA OF THE PROPOSED FLOOD CONTROL DAM AT AUBURN

The U.S. Department of Agriculture, Soil Conservation Service, published a comprehensive soil survey for Placer County in 1980. The report provides detailed pedological descriptions of each soil series in the county and limited geotechnical data for each series. The report delineates soils at a map scale of 1 inch=2000 feet. The Soil Conservation Service prepared a similar report for El Dorado County in 1974. For El Dorado County, the soils were delineated at a map scale of about 1 inch=1800 feet.

Both soil surveys were used to compile a detailed and comprehensive soils map for the reservoir area of the proposed flood control dam at Auburn (Plate 1). To compile these maps, the Placer County report was used for areas north of the Middle Fork American River and the El Dorado County report was used for the area to the south. These maps delineate soils at a scale of 1 inch=2000 feet. Soil series designations from the El Dorado County report were modified somewhat to conform to the nomenclature presented in the Placer County report.

Soil Series

Soils in the area of the proposed flood control dam at Auburn are divided into ten basic soil series. Each series defines a group of soils with similar color, texture, structure, thickness, consistency, soil profile development, and parent material. Soils series in the project area are:

Auburn	Boomer	Exchequer	Horseshoe	Inks
Josephine	Mariposa	Maymen	Sites	Sobrante

Each soil series is described below. These descriptions include the USCS (Unified Soils Classification System) designation for each. USCS designations differ from the Soil Conservation Service agricultural designations in that the USCS classifies soils on the basis of physical characteristics, such as grain-size distribution and whether the soils are plastic or nonplastic.

In the USCS, soils composed primarily of particles finer than No. 200 (0.074 mm) sieve size are classified as fine-grained soils; those composed primarily of particles coarser than No. 200 sieve size are classified as coarse-grained soils.

Fine-grained soils are divided into silt or clay depending on the soil's plasticity. Nonplastic or slightly plastic fine-grained soils are classified as silt (ML or MH); plastic fine-grained soils are classified as clay (CL or CH).

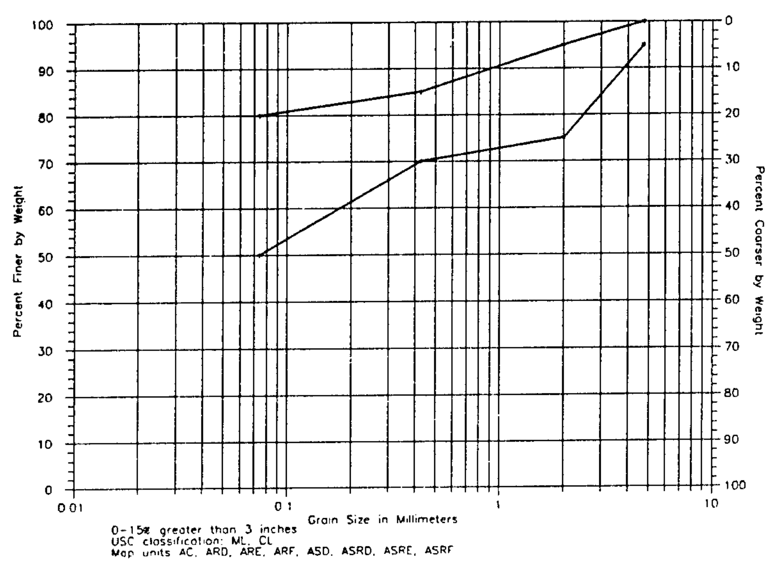
Coarse-grained soils are divided into gravel (GP or GW) or sand (SP or SW), depending on whether the soil particles are predominantly coarser than or finer than No. 4 (4.75 mm) sieve size. If coarse-grained soils contain greater than 12 percent fine-grained soils, they are classified as clayey or silty sand (SC or SM) or clayey or silty gravel (GC or GM). In the USCS, particles larger than 3 inches are not considered part of the soil.

Figures 1 through 10 show upper and lower ranges of grain-size distribution for the sand-size particles for each soil series. The graphs were produced from data in the Soil Conservation Service reports for Placer and El Dorado counties.

Auburn Soil Series

The Auburn series is common in the lower reaches of the project area. These are shallow, well drained soils, generally underlain by metavolcanic rock at depths of 12 to 28 inches. These soils typically develop on slopes ranging from 2 to 70 percent. Coarse-grained fragments within the soil profile range from few to about 25 percent by volume and consist of gravels, cobbles, and larger stones. Contact with the underlying bedrock is generally abrupt. The USCS designation for the Auburn soil series is typically nonplastic silt (ML).

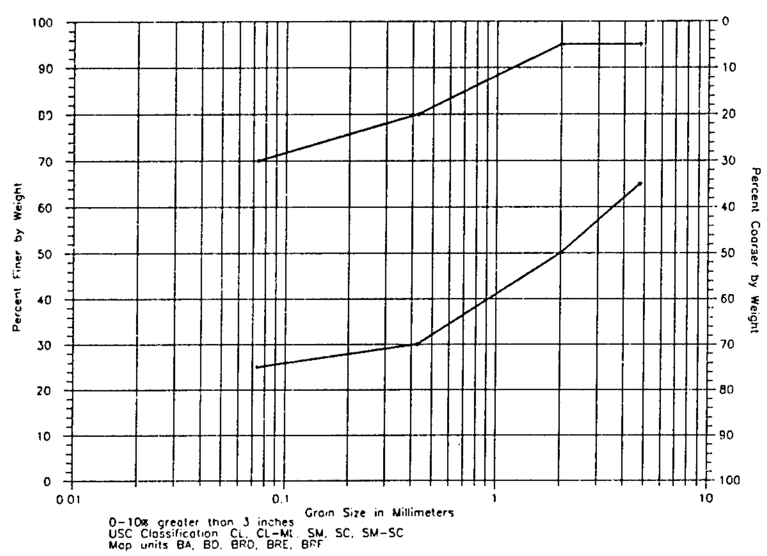
Figure 1
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR AUBURN SOIL SERIES



Boomer Soil Series

The Boomer series is somewhat common in the lower reaches of the proposed project area but confined mainly to north-facing slopes. The Boomer series consists of deep, well drained soils underlain by metavolcanic rock. In the proposed reservoir area, Boomer soils develop on slopes ranging from 2 to 70 percent, primarily on deeply weathered bedrock or older landslide masses. Depth to bedrock ranges from 40 to 60 inches, or more. Gravels less than 1 inch in diameter make up 5 to 35 percent of the soil by volume, with the volume of gravel increasing with depth. USCS designations for the Boomer soil series are lean clay (CL), lean clay-silt (ML-CL), or clayey sand (SC).

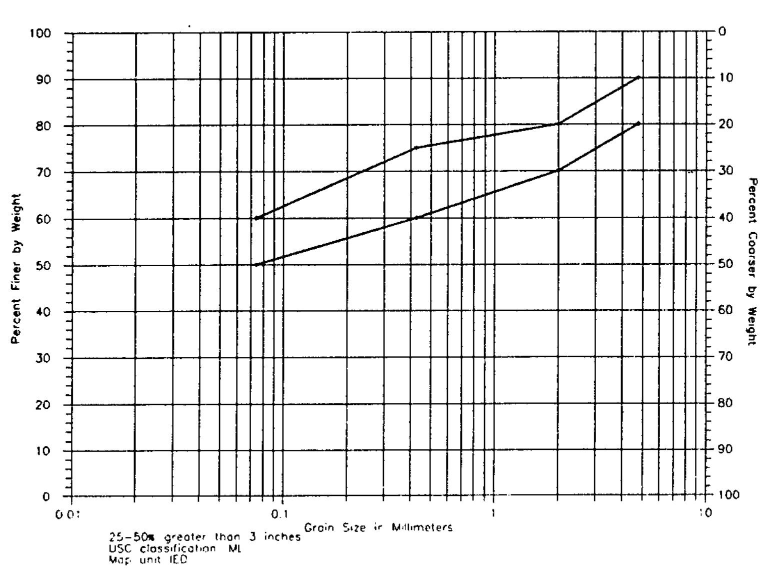
Figure 2
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR BOOMER SOIL SERIES



Exchequer Soil Series

The Exchequer series is associated with the Inks series in the project area and is found only locally on the uppermost slopes near the proposed damsite. This soil series consists of shallow, somewhat excessively drained soils underlain by hard andesitic breccia. These soils form on the tops and sides of volcanic capped ridges at slopes of 2 to 30 percent. Depth to bedrock ranges from 8 to 20 inches. Gravel and large cobbles cover 1 to 5 percent of the surface and comprise 10 to 25 percent of the soil column. The USCS designation for the Exchequer series is gravelly inorganic silt (ML).

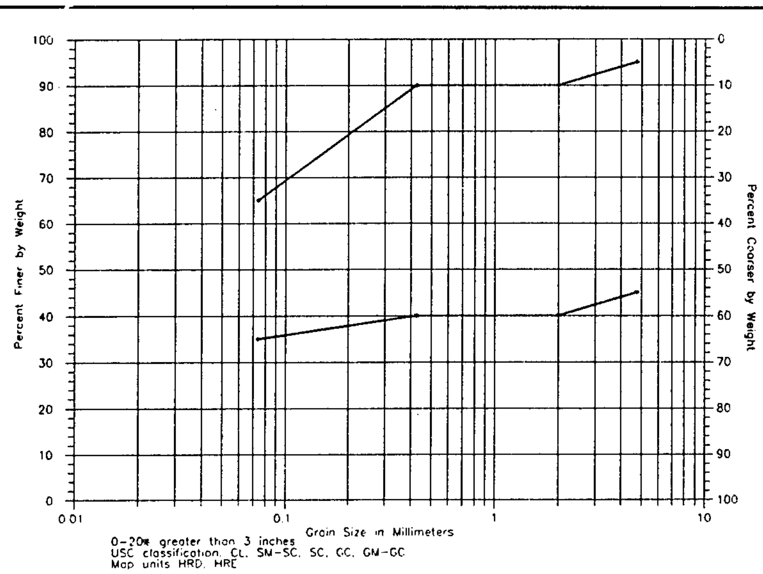
Figure 3
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR EXCHEQUER SOIL SERIES



Horseshoe Soil Series

The Horseshoe series is found only at a few locations in the upstream reaches of the project area. The soil series consists of very deep, well drained soils that formed on old alluvial river deposits of mixed sources. The older alluvium is high in content of gravel and cobble-sized quartz, chert, and other resistant minerals and rock types. Slopes range from 2 to 30 percent. USCS designations for the Horseshoe series are clayey sand (SC), silty sand-clayey sand (SM-SC), clayey gravel (GC), silty gravel-clayey gravel (GM-GC), and lean clay (CL)

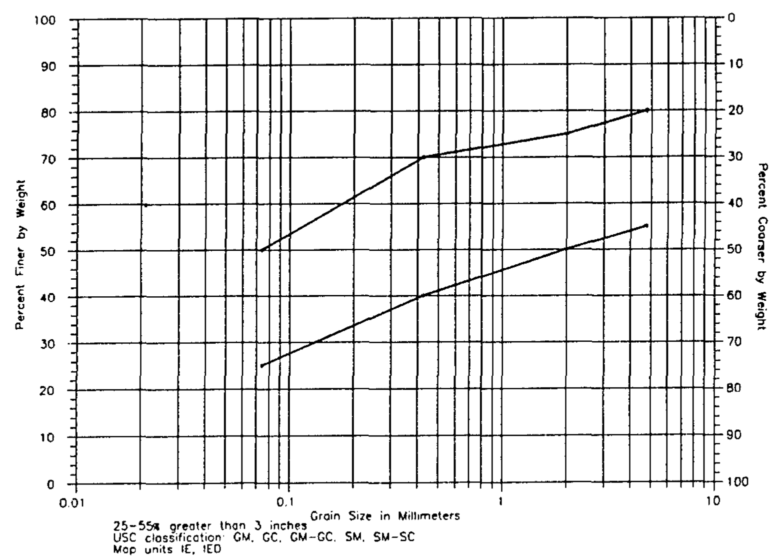
Figure 4
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR HORSESHOE SOIL SERIES



Inks Soil Series

The Inks series is associated with the Exchequer series in the project area. The Inks series consists of shallow, well drained soils underlain by andesitic conglomerate at depths of 12 to 20 inches. Slopes range from 2 to 50 percent. Inks soils typically contain coarse fragments from 15 to 50 percent. USCS designations for the Inks series are silty gravel (GM), silty sand (SM), silty gravel-clayey gravel (GM-GC), silty sand-clayey sand (SM-SC), clayey gravel (GC), and clayey sand (SC).

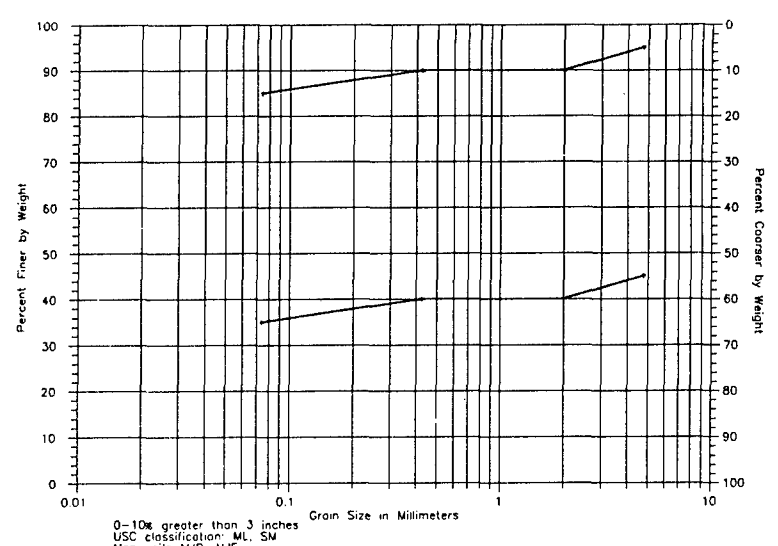
Figure 5
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR INKS SOIL SERIES



Josephine Soil Series

The Josephine series is associated with the Mariposa series and is found only in a small area in the middle reaches of the project area. Josephine soils form primarily on south-facing slopes and are deep, well drained soils underlain by weathered metamorphic bedrock or old landslide masses on slopes of 2 to 70 percent. Depth to soft, strongly weathered metamorphic rock ranges from 40 to 60 inches, or more. The series often contains 5 to 15 percent gravel less than 1 inch in diameter and occasionally contains cobbles. USCS designations for the Josephine series are silt (ML) and silty sand (SM).

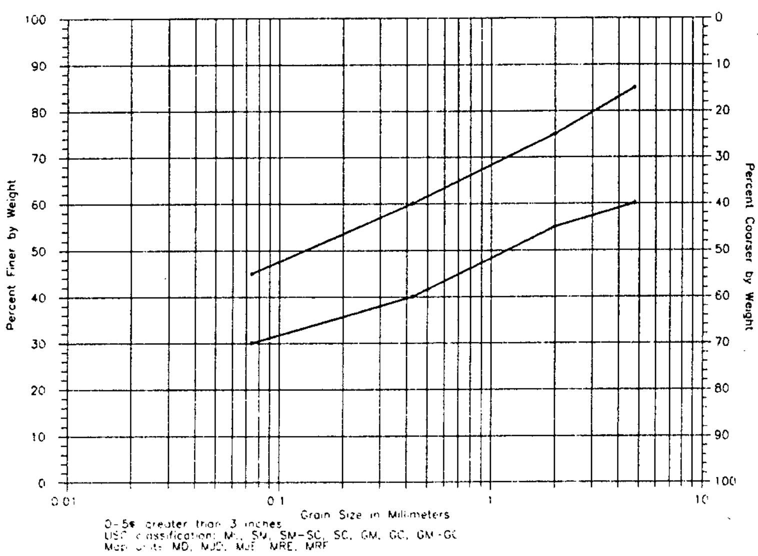
Figure 6
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR JOSEPHINE SOIL SERIES



Mariposa Soil Series

The Mariposa series consists of shallow to moderately deep, well drained soils underlain by highly fractured, vertically foliated schist or slate in the middle reaches of the project area. Depth to weathered slate or schist ranges from 15 to 35 inches. Gravel and cobbles comprise about 15 to 30 percent of the soil column by volume. USCS designations for the Mariposa series are silty sand (SM), silty gravel (GM), silty sand-clayey sand (SM-SC), clayey sand (SC), clayey gravel (GC), and silty gravel-clayey gravel (GM-GC).

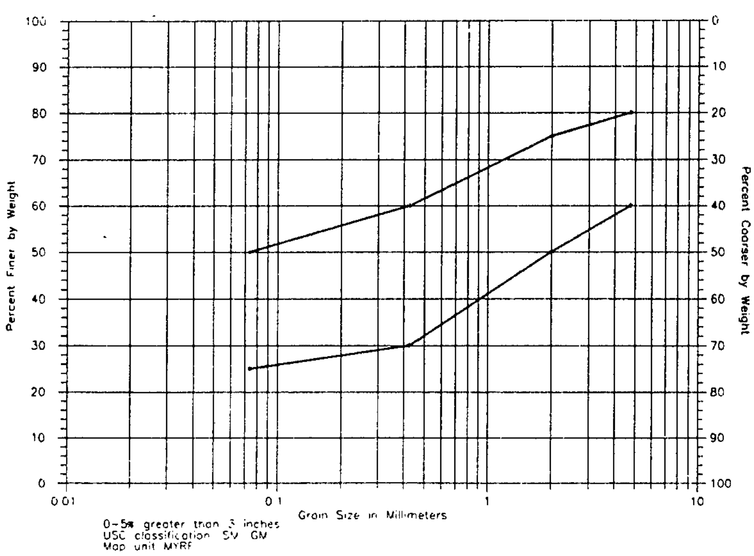
Figure 7
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR MARIPOSA SOIL SERIES



Maymen Soil Series

The Maymen series consists of shallow, somewhat excessively drained soils underlain by hard metamorphic rock. These soils are found in the uppermost reaches of the project area. The series develops on slopes ranging from 9 to 75 percent and forms a veneer over hard slate about 8 to 20 inches thick. Gravels and cobbles are 15 to 30 percent of the soil profile by volume. USCS designations for the Maymen series are silty sand (SM) and silty gravel (GM).

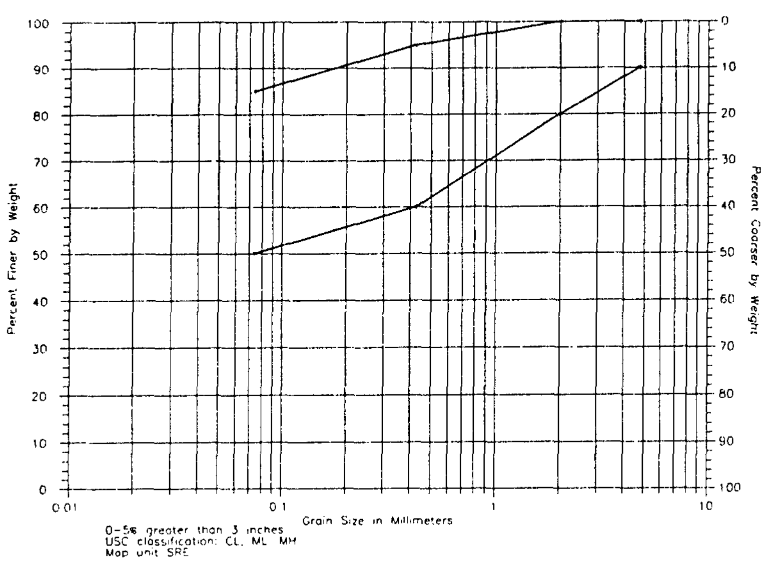
Figure 8
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR MAYMEN SOIL SERIES



Sites Soil Series

The Sites series is found at only a few scattered locations on the upper slopes of ridges in the middle reaches of the proposed reservoir area. The series consists of deep, well drained soils underlain by slate, schist, intrusive igneous rock, or old landslide masses. The soil develops on slopes ranging from 2 to 50 percent. Depth to soft schistose rock ranges from 40 inches to more than 7 feet and is more than 60 inches in most areas. The soil profile contains 5 to 30 percent gravel or gravel-size fragments of slate. USCS designations for the Sites series are silt (ML), lean clay (CL), and elastic silt (MH).

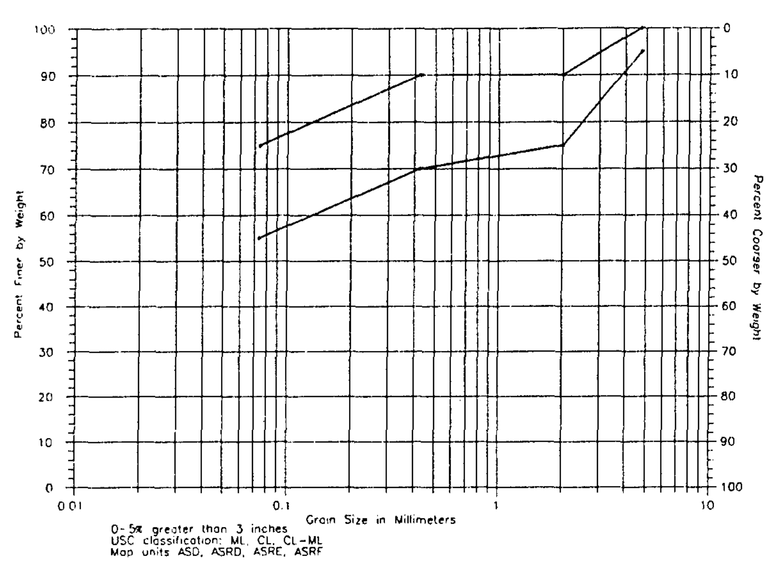
Figure 9
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR SITES SOIL SERIES



Sobrante Soil Series

The Sobrante series is associated primarily with other soil series in the downstream reaches of the project area and consists of moderately deep, well drained soils underlain by mafic schist on slopes of 2 to 70 percent. Depth to bedrock ranges from 22 to 40 inches. Coarse fragments are 3 to 15 percent of the soil column by volume. Most of these fragments are either near the surface or directly above bedrock. USCS designations for the Sobrante series are silt (ML), lean clay (CL), and lean clay-silt (CL-ML).

Figure 10
UPPER AND LOWER RANGE OF GRAIN-SIZE DISTRIBUTION FOR SOBRANTE SOIL SERIES



Riverwash and Rock Land

In addition to the soil series described above, Plate 1 identifies areas of river alluvium (Riverwash) and large rock outcrop areas (Rock land).

Riverwash occurs in and adjacent to the North and Middle Forks of the American River. The material is highly stratified stoney and bouldery sand, which is subject to scour and deposition depending on streamflow velocities and stream bedload. Riverwash material is typically barren of vegetation.

Areas identified as Rock land are outcrops of highly resistant metamorphic rock, andesite, and serpentinite. These rocks crop out mainly on steep to very steep slopes that break into major drainage ways. About 10 to 50 percent of the rock outcrops are covered with a very thin mantle of soil.

Soil Series Gradations

The grain-size distribution curves (Figures 1 through 10) show that Auburn, Exchequer, Sites, and Sobrante soil series are fine-grained soils. Soils of all four are primarily nonplastic silts, although Auburn, Sites, and Sobrante soils grade locally to a clay of low to medium plasticity. The Mariposa, Maymen, and Inks series are all coarse-grained soils but contain a relatively high percentage of plastic or nonplastic fine-grained soils. The Horseshoe, Josephine, and Boomer series vary from fine-grained to coarse-grained soils that contain a high percentage of plastic or nonplastic fine-grained soils.

Examination of the grain-size distribution curves shows soils in the downstream reaches of the proposed reservoir site are distinctively different than those in the middle and upstream reaches. Soils in the downstream reservoir reaches are consistently more fine-grained than soils in the middle and upstream reaches.

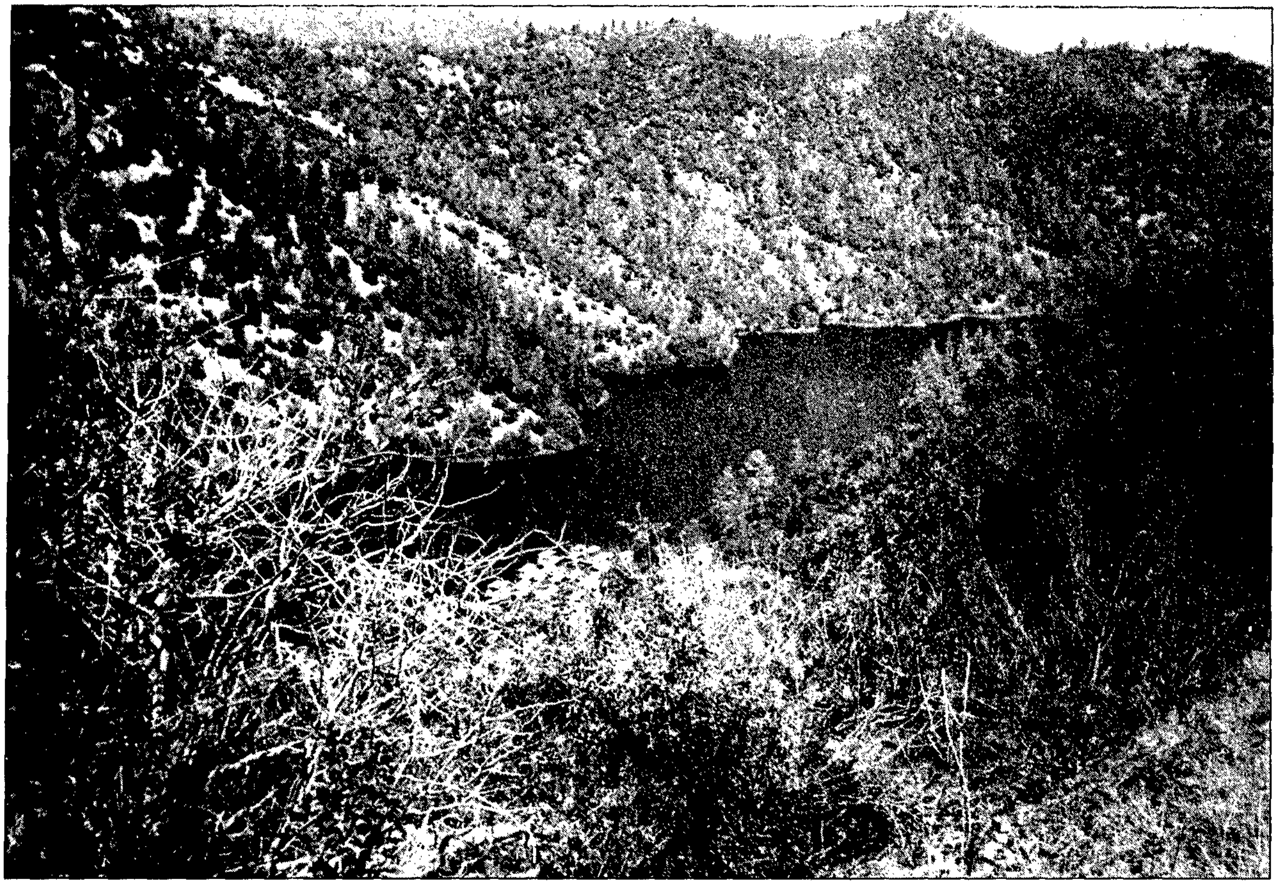
Soil Complexes

At several places in the proposed project area, individual soil series occur in such small and intricate patterns that they cannot be shown separately at the map scale. Soils in these areas are grouped and identified as a *soil complex* by the Soil Conservation Service. Within the project area, 10 soil complexes are shown on Plate 1. These are:

Auburn-Rock outcrop	Auburn-Sobrante silt loam
Auburn-Sobrante-Rock outcrop	Boomer-Rock outcrop
Horseshoe-Rubble land	Inks-Exchequer
Mariposa-Josephine	Mariposa-Rock outcrop
Maymen-Rock outcrop	Sites-Rock outcrop

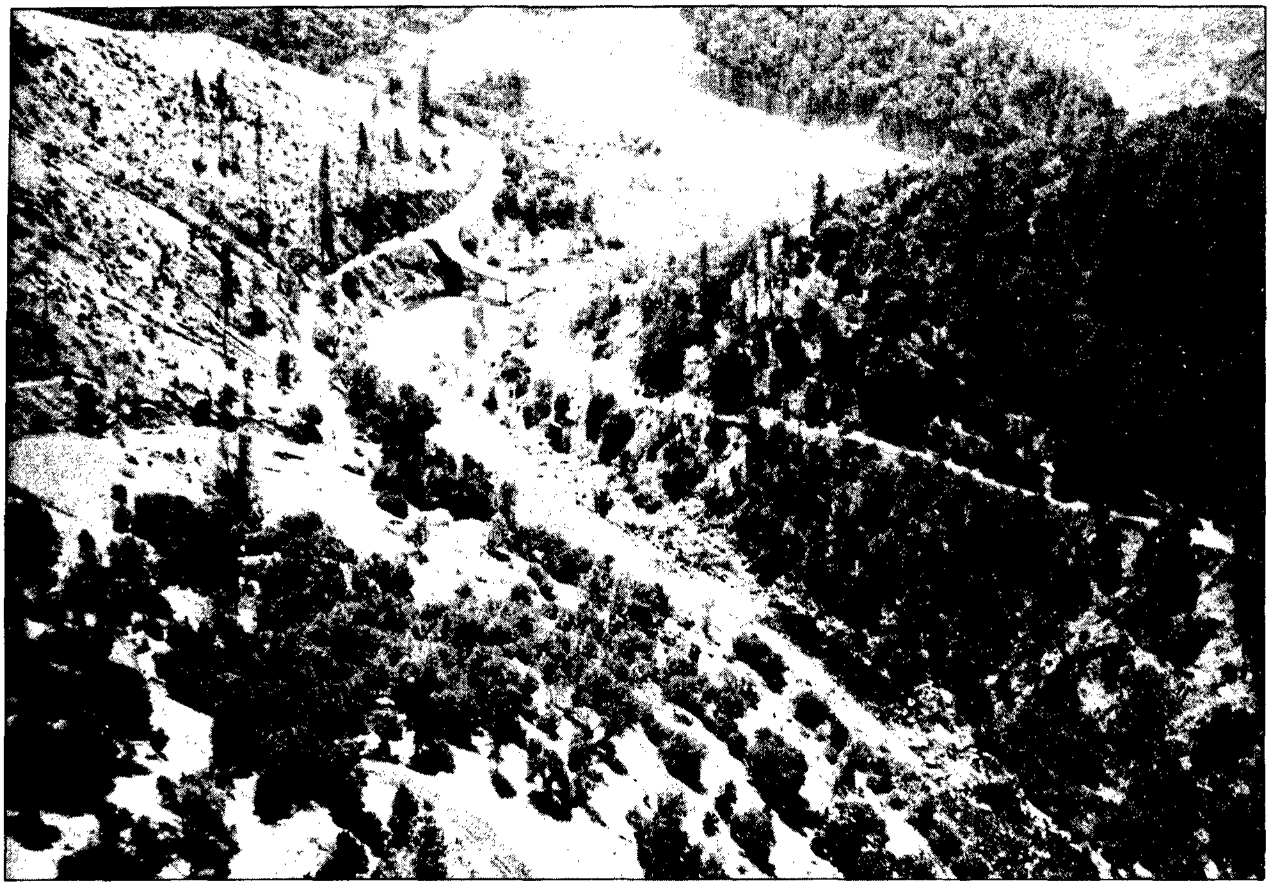
Auburn-Rock Outcrop Complex

The Auburn-Rock outcrop complex consists of 60 percent Auburn soils and 15 percent metamorphic rock outcrop. The remaining 25 percent is composed of Sobrante silt loam and Boomer loam, mainly on side slopes facing north and east. Rock outcrop areas consist of hard metavolcanic rock, schist, or slate covering areas up to 100 square feet. Some outcrops are 1 to 2 feet high.



Auburn-Sobrante-Rock Outcrop complex on the north shore of Lake Clementine.

Auburn-Sobrante-Rock Outcrop complex on slopes adjacent to the North Fork American River, downstream from Lake Clementine.



North Fork American River near the confluence with the Middle Fork.
Auburn-Sobrante-Rock Outcrop complex on the left slope; Boomer-Rock Outcrop on the right slope.

Auburn-Sobrante Silt Loam on slopes adjacent to the Middle Fork American River.
Note Riverwash materials in canyon bottom.

Auburn-Sobrante Silt Loam

The Auburn-Sobrante silt loam complex is composed of 50 percent Auburn soil and 40 percent Sobrante soil. About 8 percent of the unit includes areas of Boomer loam, mainly on the north- and east-facing slopes, and 2 percent is scattered rock outcrops.

Auburn-Sobrante-Rock Outcrop Complex

The Auburn-Sobrante-Rock outcrop complex is composed of areas underlain by 30 to 45 percent Auburn soil, 25 to 30 percent Sobrante soils, and 12 to 20 percent metamorphic bedrock outcrop. The remaining 13 to 25 percent is composed of Boomer soils, primarily confined to sideslopes facing north and east. The metamorphic bedrock outcrops cover areas up to 500 square feet and commonly stand 1 to 2 feet high. The percent of exposed metamorphic rock in this complex increases on steeper slopes.

Boomer-Rock Outcrop Complex

The Boomer-Rock outcrop complex is about 55 to 60 percent Boomer soil and 10 to 15 percent metamorphic rock outcrop. About 5 to 10 percent of the unit includes areas of Josephine loam, 10 to 15 percent Sobrante loam, 0 to 5 percent Sites loam, and 5 to 10 percent Auburn silt loam. Rock outcrop consists of areas of scattered hard metamorphic rock 1 to 2 feet high. Some of the rock outcrops cover areas of 100 square feet or more.

Boomer soil series on slopes adjacent to the Middle Fork American River.
Auburn-Sobrante silt loam in foreground.

Horseshoe-Rubble Land Complex

The Horseshoe-Rubble land complex develops on Tertiary age river terrace deposits and their sideslopes. The unit is about 45 percent Horseshoe soil and 40 percent rubble land. The Horseshoe soil adjoins and is often isolated by deep, vertically walled hydraulic mine pits. Rubble land is the material left in the bottom of the pit. The remaining 15 percent of the unit is composed of Josephine soils.

Inks-Exchequer Complex

The Inks-Exchequer complex is about 40 percent Inks soil and 30 percent Exchequer soil. About 20 percent of the soil is similar to the Exchequer soil but is either shallow or has a loam subsoil. About 10 percent of the unit includes areas that are similar to the Inks soil but that lack cobbles in the subsoil and are 12 to 26 inches deep to bedrock.

Mariposa-Josephine Complex

The Mariposa-Josephine complex is about 55 percent Mariposa soil and 35 percent Josephine soil. Generally, the Mariposa soil is on the ridges and sharp breaks, and on south- and west-facing sideslopes. In some places Josephine soil develops on concave slopes. In other places it occupies smooth north- and east-facing sideslopes. About 8 percent of the unit includes areas of Sites soils, and 2 percent is scattered areas of rock outcrops.

Mariposa-Rock Outcrop Complex

The Mariposa-Rock outcrop complex is composed of about 60 to 65 percent Mariposa soil and 10 to 15 percent scattered outcrops of metamorphic rock. Some rock outcrops are larger than 1/2 acre. About 10 to 15 percent of the unit is Josephine loam, 5 to 10 percent is Maymen gravelly loam, and 5 percent is Sites loam.

Maymen-Rock Outcrop Complex

The Maymen-Rock outcrop complex is 45 to 50 percent Maymen soils and 20 to 25 percent rock outcrop. About 25 percent of this unit includes areas of Mariposa gravelly loam, and 5 percent is Josephine loam. Rock outcrops occur in scattered areas of exposed metamorphic rock, and some outcrops cover more than 5 acres.

Sites-Rock Outcrop Complex

Sites-Rock outcrop complex is about 60 percent Sites soils and 15 percent metamorphic rock outcrop. The unit includes areas of about 15 percent Josephine loam, 5 percent Mariposa gravelly loam, and 5 percent a soil similar to the Sites soil but 30 to 40 inches deep.

Slope Designation

On Plate 1, both soil series and soil complexes are further subdivided based on slope. The last letter in the soil series or complex designation identifies the percent slope on which the soil occurs. The percent slopes are designated as follows:

Letter Designation	Percent Slope
A	Nearly level
B	< 9 percent
C	< 15 percent
D	< 30 percent
E	< 50 percent
F	< 70 percent

Example: BD = Boomer soil series on slopes of 30 percent or less.

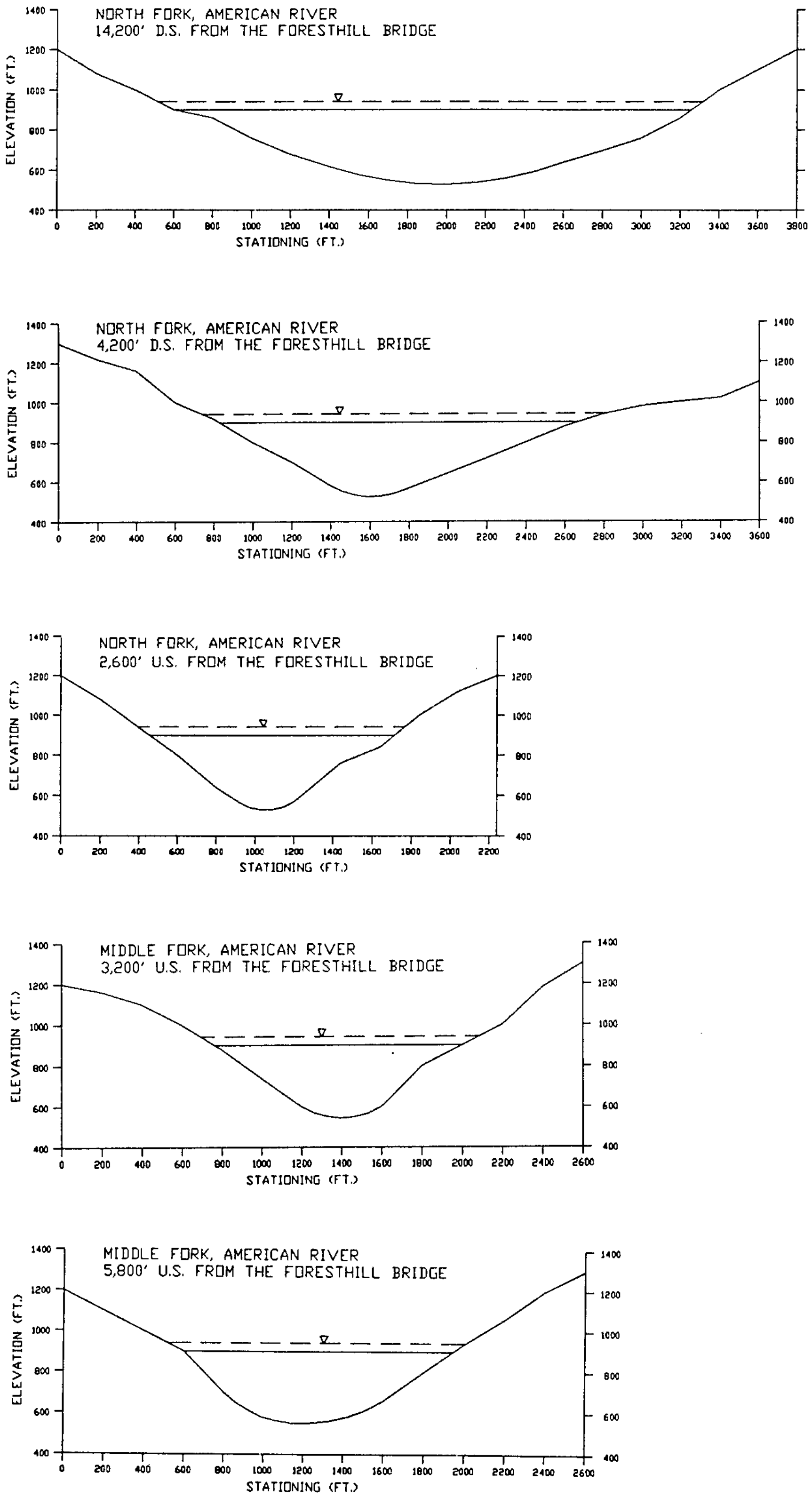
Examination of Plate 1 shows that almost all the side slope soils in the proposed project area are on slopes ranging from 50 to 70 percent. Figure 11 shows typical slopes at several sections across the reservoir area.

Soils Within the Inundation Zone of the Proposed Flood Control Dam at Auburn

Not all the soil series or soil complexes in the project area lie within the inundation zone for the proposed 200- or 400-year flood control dam. Table 1 shows soil series and soil complexes in the project area and the approximate percentage of each that would be within the inundation zone. These percentages were estimated by planimetering the total area of each soil series or soil complex and dividing each by the total inundation area for the 400-year flood control dam. The inundation zone was defined as all areas upstream from the proposed dam and below elevation 942. The percentages are roughly identical for the 200- and 400-year reservoirs.

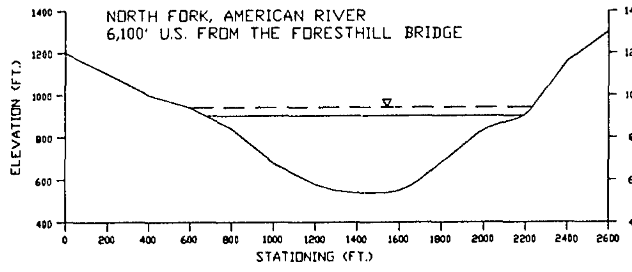
Table 1 shows that Riverwash and Rock land occupy more than 35 percent of the inundation zone for the proposed 200- and 400-year flood control dam. The Riverwash soils lie on the canyon floors and are continually being reworked by both forks of the American River. Rock land is an area of hard metamorphic rock outcrop. Neither unit should be affected by periodic inundation.

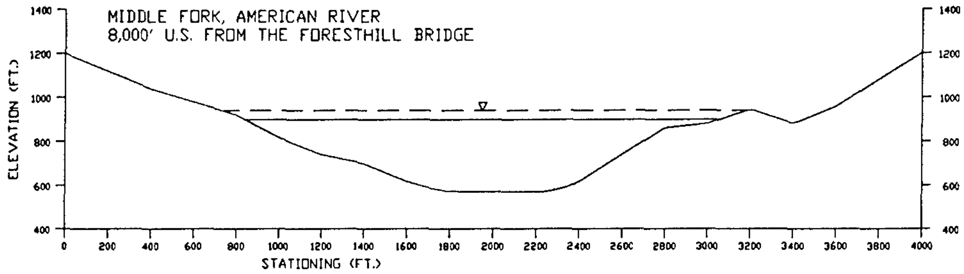
Figure 11
TYPICAL CROSS SECTIONS OF THE CANYON IN THE AREA OF THE PROPOSED FLOOD CONTROL DAM AT AUBURN

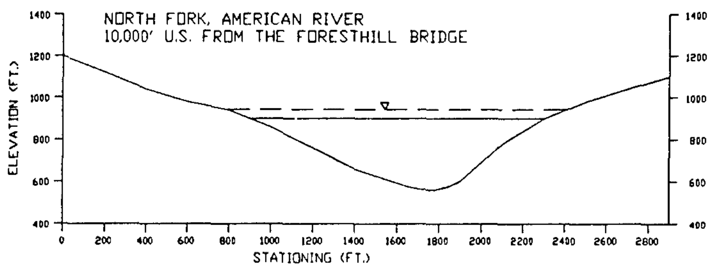


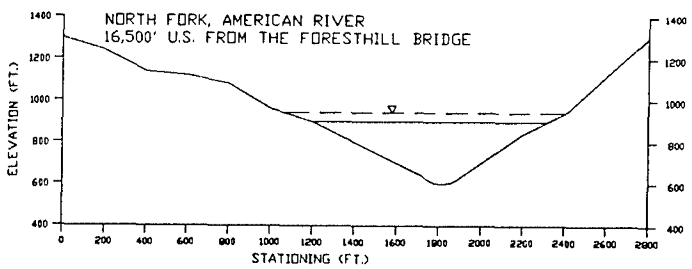
NOTES:
All cross sections are oriented looking downstream.
All elevations and distances are taken from USGS 7.5-minute quad maps.
——— Water surface at the spillway elevation for the proposed 200-year facility.
- - - - Water surface at the spillway elevation for the proposed 400-year facility.

Figure 11 (continued)
TYPICAL CROSS SECTIONS OF THE CANYON IN THE AREA OF THE PROPOSED FLOOD CONTROL DAM AT AUBURN









NOTES:
All cross sections are oriented looking downstream.
All elevations and distances are taken from USGS 7.5-minute quad maps.
——— Water surface at the spillway elevation for the proposed 200-year facility.
– – – – Water surface at the spillway elevation for the proposed 400-year facility.

Table 1
SOIL SERIES AND SOIL COMPLEXES WITHIN THE INUNDATION ZONE FOR THE PROPOSED 200- AND 400-YEAR FLOOD CONTROL DAMS AT AUBURN

		Percent of Total Inundation Area	
	Soil Series or Complex	By Soil and Slope	By Soil
AC	Auburn, silt loam, 2-15% slopes	—	—
ARD	Auburn, rock outcrop complex, 2-30% slopes	0.4	
ARE	Auburn, rock outcrop complex, 2-50% slopes	2.8	
ARF	Auburn, rock outcrop complex, 2-70% slopes	3.2	6.4
ASD	Auburn, Sobrante silt loams, 15-30% slopes	—	—
ASRD	Auburn, Sobrante, rock outcrop complex, 2-30% slopes	0.6	
ASRE	Auburn, Sobrante, rock outcrop complex, 30-50% slopes	1.8	
ASRF	Auburn, Sobrante, rock outcrop complex, 50-70% slopes	23.7	26.1
BA	Boomer loam, 2-5% slopes	—	
BD	Boomer loam, 15-30% slopes	0.6	0.6
BRD	Boomer, rock outcrop complex, 5-30% slopes	0.3	
BRE	Boomer, rock outcrop complex, 30-50% slopes	3.5	
BRF	Boomer, rock outcrop complex, 50-70% slopes	10.9	14.7
HRD	Horseshoe, rubble land complex, 2-30% slopes	—	
HRE	Horseshoe, rubble land complex, 30-50% slopes	—	—
IE	Inks, cobbly loam, 30-50% slopes	—	—
IED	Inks, Exchequer complex, 2-25% slopes	—	—
MD	Mariposa, gravelly loam, 5-30% slopes	—	—
MJD	Mariposa, Josephine complex, 5-30% slopes	—	
MJE	Mariposa, Josephine complex, 30-50% slopes	0.4	0.4
MRE	Mariposa, rock outcrop complex, 5-50% slopes	0.2	
MRF	Mariposa, rock outcrop complex, 50-70% slopes	8.8	9.0
MYRF	Maymen, rock outcrop complex, 50-75% slopes	5.7	5.7
SRE	Sites, rock outcrop complex, 15-50% slopes	0.6	0.6
Rl	Rock land	8.3	8.3
R	Riverwash	27.7	27.7
	Quarry and Rubble land	0.6	0.6

SOIL STABILITY

Several factors determine stability of soils in the reservoir area for the proposed flood control dam:

- Slope on which the soil resides,
- Overall shear strength of the soil,
- Thickness of the soil column,
- Unit weight of the soil, and
- Degree of saturation.

The forces attempting to mobilize the soil mass downhill are directly proportional to the slope, unit weight, and thickness of soil. The shear strength developed along the failure surface provides resistance to this driving force and attempts to keep the soil in place. When the driving force exceeds the resisting force, the soil mass will mobilize.

When the soil mass on the hillside becomes saturated, its stability is affected in two ways.

- First, the intergranular neutral stress exerted by the water reduces the overall shear strength along the failure surface.

- Second, the additional weight of water filling the soil's pore spaces increases the overall weight of the soil mass and, thus, increases the down-slope driving force.

As the soil mass drains, the intergranular neutral stress is reduced, increasing the shear strength along the failure surface, and the unit weight of the soil decreases, reducing the down-slope driving force.

The numerous shallow slope failures that resulted from the Auburn cofferdam failure in 1986 suggest that, of all the factors discussed above, the degree of saturation was probably the most critical factor with respect to the soil's stability. When water was stored in the reservoir before the cofferdam failed, all the soils below the high water elevation were probably totally saturated. Although the shear strength of the soil was reduced by the saturation, the loss in shear strength was offset by a reduction in the unit weight of the soil column from buoyancy. When the dam was breached by floodwaters, the reservoir drained in less than an hour (J. Burke, U.S. Bureau of Reclamation, personal communication), not allowing enough time for the soils to drain. Therefore, almost completely saturated soils were exposed on the hillsides. The sudden increase in unit weight because of loss of buoyancy, and the decrease in shear strength because of the soil's saturated condition most likely initiated the failures.

Under normal operation of the proposed flood control dam, stability of soils in the reservoir area will probably be directly related to the rate at which the soil column drains in relation to the rate the reservoir lowers (vertical phreatic lag). Soils that drain almost as fast as the drop in reservoir head will remain stable; soils that have a long vertical phreatic lag will probably mobilize if they are on steep enough slopes.

To estimate the vertical phreatic lag that would be required to mobilize soils in the reservoir area (critical phreatic lag), the height of slide scars that resulted from the 1986 cofferdam failure were measured and evaluated. The slide scars represent the failure surfaces along which the soil mobilized in a totally saturated condition. Slide scar heights were estimated from maps (scale 1 inch = 2,000 feet), provided by the U.S. Army Corps of Engineers, that show many of the slope failures resulting from the 1986 cofferdam breach. Slope failure that could be directly attributed to undercutting of the soil mass were excluded from the evaluation. The scar height was assumed to be roughly half the total vertical distance from the toe of the displaced soil mass to the top of the scar. A more

refined estimate could not be made because the Corps of Engineers mapping included both the slide scar and displaced soil mass as one unit.

Figure 12
HISTOGRAM OF SLOPE FAILURE SCAR HEIGHTS
FOLLOWING FAILURE OF AUBURN COFFERDAM

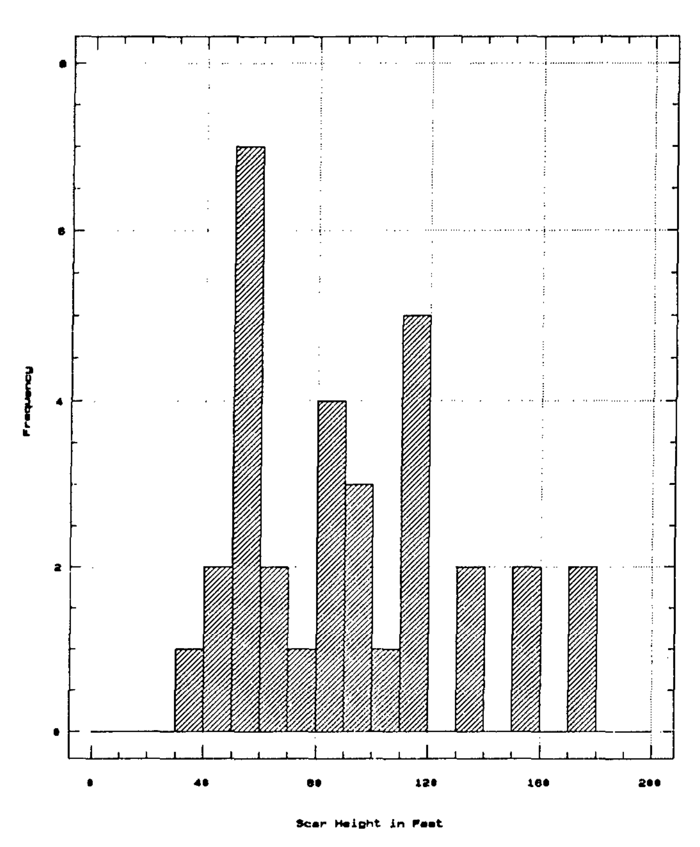


Figure 12 presents the scar height data in the form of a histogram. It can be concluded from these data that soils in the inundation zone should remain stable as long as the vertical phreatic lag in the soil column does not exceed about 35 feet. Figure 12 also shows that most scar heights in the reservoir area were in the 50- to 120-foot range, so concluding that slope failures would occur with as little as a 35-foot phreatic lag is conservative.

To evaluate how normal operation of the proposed flood control dam at Auburn would affect stability of the soils, calculated drainage rates of the reservoir soils were compared with drawdown rates for the reservoir. Based on reservoir drawdown curves provided by the U.S. Army Corps of Engineers for a 200-year and 400-year flood control dam, the drawdown rates were determined as shown in Table 2.

Average drawdown rate was determined by the total change in reservoir head divided by the total time necessary to evacuate the reservoir. Maximum drawdown rates were determined by calculating the maximum slopes of all drawdown curves provided by the Corps of Engineers. In almost every case, the maximum drawdown rate for the reservoir occurs after it has receded to less than half its peak value. For comparison, the average drawdown rate in the reservoir behind the cofferdam after it failed was greater than 200 feet/hour.

Table 2
RESERVOIR DRAWDOWN RATES FOR
PROPOSED 200- AND 400-YEAR FLOOD CONTROL DAMS AT AUBURN
(feet/hour)

	Year Storm					
400-Year Facility	400	200	100	50	25	10
Average Drawdown	0.97	1.06	1.14	1.13	1.06	1.65
Maximum Drawdown	1.26	1.61	1.92	1.67	1.22	1.92
		Year Storm				
200-Year Facility		200	100	50	25	10
Average Drawdown		1.91	2.06	2.32	2.91	4.33
Maximum Drawdown		3.75	3.75	3.49	4.17	6.60

GROUND WATER FLOW MODEL

To estimate the vertical phreatic lag that would occur in the reservoir soils as a result of receding reservoir levels, the U.S. Geological Survey's 3-dimensional finite-difference ground water flow model MODFLOW (McDonald and Harbaugh, 1983) was used to evaluate soil drainage. Most sideslope soils in the area are on slopes ranging from 50 to 70 percent, so two models were developed, one for soils on 70 percent slopes and one for soils on 50 percent slopes. Both models simulate unconfined gravity flow in a sloping soil column that would result from reservoir lowering. Both models represent an average soil column along a 3-1/2-foot-high section of slope. Figures 13 and 14 show the discretization of cells for the two models.

Soils on 50 percent slopes were modeled with a single row of 143 cells. Each cell is 1 inch square horizontally, with its bottom elevation specified in a stair-step fashion to approximate a 50 percent slope between the soil and the underlying rock. In the simulation, it was assumed that no water moved into or out of the rock. The initial head in cells 1-59 was set at a constant value to represent the horizontal phreatic surface caused by the reservoir when it crests. The initial head in cells 60-142 was reduced incrementally in a stair-step fashion to approximate a 30-inch column of saturated soil above the rock. The head in the last cell was held constant through the simulation to provide drainage for the model.

To intercept and account for water that would drain to the sloping portion of the ground surface, a series of high conductance model drains was specified at the initial head elevation in the last 84 cells. The model simulates 2 hours of drainage under transient conditions. Heads for each cell were printed at each 10-minute time-step to evaluate changes in drainage over time.

The 70 percent slope model was implemented in the same way, except that only 102 cells were needed and proportionally more cells were used to model the sloping portion of the soil column and fewer cells were used to model the horizontal phreatic surface.

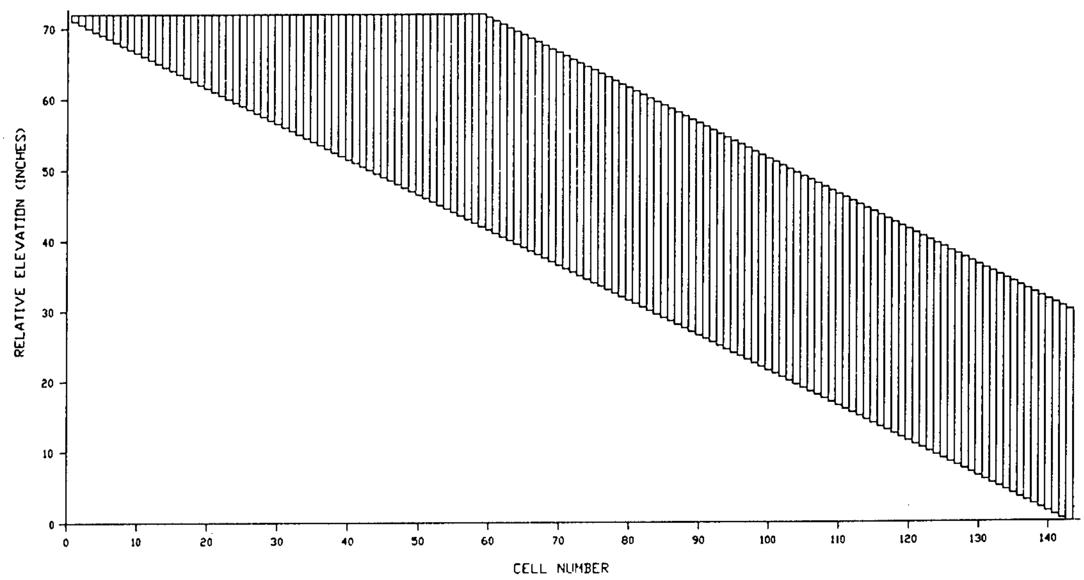
Soil Permeability

Permeability values used in the models were taken from the Soil Conservation Service reports for Placer and El Dorado counties. Table 3 summarizes the permeability ranges for sideslope soils within the inundation zone for the 200- and 400-year flood control dam. Sideslope soils within the inundation zone fall within two permeability ranges that average 0.0233 inches per minute for the higher permeability Auburn, Mariposa, Mayman, and Sobrante soil series and 0.0066 inches per minute for the lower permeability Boomer, Sites, and Josephine soil series. The storage coefficient for all soil series used in both models was 0.03.

Table 3
PERMEABILITY OF SOIL SERIES IN THE AREA OF THE PROPOSED FLOOD CONTROL DAM AT AUBURN
(inches/minute)

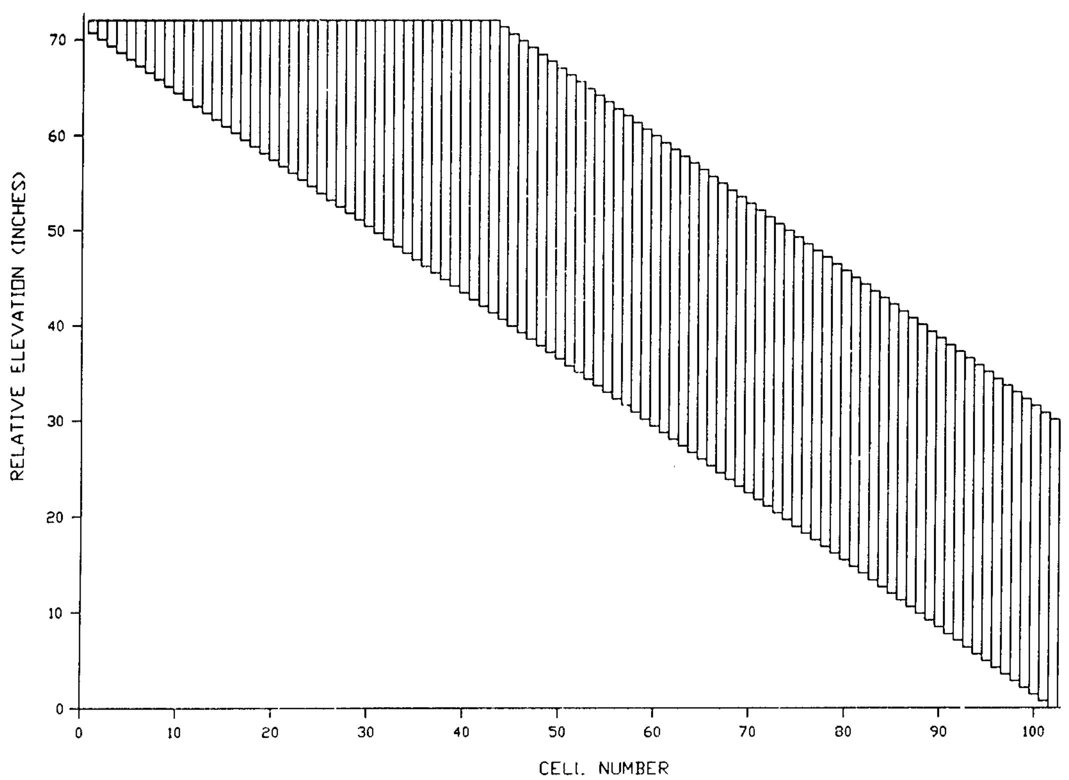
0.033 to 0.010	0.010 to 0.0033
Auburn	Boomer
Mariposa	Sites
Mayman	Josephine
Sobrante	

Figure 13
DISCRETIZATION OF 50 PERCENT SLOPE MODEL



NOTES:
Top elevation of each cell indicates the initial head used for each cell in the model.
Drains for cells 60 to 142 were set at the initial head elevation for each cell.
Cell 143 is specified as a constant head cell in the model.

Figure 14
DISCRETIZATION OF 70 PERCENT SLOPE MODEL



NOTES:
Top elevation of each cell indicates the initial head used for each cell in the model.
Drains for cells 44 to 101 were set at the initial head elevation for each cell.
Cell 102 is specified as a constant head cell in the model.

Model Results

The percentage of drainage was determined by dividing the total volume of unsaturated soil by the total model volume above the specified constant head. Results for both models are summarized on Figures 15 and 16. The model outputs are on file with the Geology and Ground Water Section, Central District, California Department of Water Resources.

In the 50 percent slope model, the higher permeability soils drained about 50 percent at the end of the 2-hour simulation. For the lower permeability soils, only about 15 percent of the soil column had drained during the same period.

During the 2-hour simulation period with the 70 percent slope model, the soil column for the higher permeability soils had drained about 70 percent and for the lower permeability soils had drained only about 45 percent.

An evaluation of the volumetric budget from the model revealed that for the 50 percent slope model, about 2 percent of the total drainage occurred as a result of seepage from the soil to the sloping ground surface for the higher permeability soils. This value increased to about 9 percent for the lower permeability soils. In the 70 percent slope model, no surface seepage occurred for either the higher or lower permeability soils.

Results from the model show that the average vertical drainage rate for the higher permeability soils is about 0.7 foot per hour on 50 percent slopes and about 0.8 foot per hour on 70 percent slopes. These higher permeability soils include the Auburn, Mariposa, Maymen, and Sobrante soil series, which comprise about 50 percent of the soils series within the inundation zone. Vertical drainage rates were estimated by subtracting the average model head after 1 hour from the average model head after 2 hours.

If the proposed flood control reservoir is drawn down at or less than this rate, then little or no vertical phreatic lag should develop in the soil column. If no vertical phreatic lag develops, the soils will remain stable. The drawdown rate probably can be increased somewhat, to about 1 foot per hour, without causing soil stability problems because there would be insufficient time for the vertical phreatic lag to reach a critical value before the reservoir was completely drained.

A more precise maximum drawdown rate — one that will not exceed the critical phreatic lag value — cannot be computed until detailed field studies are completed to accurately measure representative permeabilities for each soil series in the inundation zone.

The model also shows that the drainage rate for the lower permeability soils is much slower — about 0.4 foot per hour on 50 percent slopes and about 0.5 foot per hour on 70 percent slopes. The lower permeability soils in the inundation zone include the Boomer, Josephine, and Sites soil series, which comprise about 15 percent of the inundation zone.

The proposed average drawdown rates for a 400-year flood control dam are very near the estimated maximum 1 foot per hour maximum value determined for the higher permeability soils, so they should remain stable under these drawdown conditions. Proposed drawdown rates for a 200-year flood control dam are nearly twice those of the 400-year dam, so soil stability might be a problem under these drawdown conditions. The drainage rate for the lower permeability soils is such that, at drawdown rates specified for the 200- or 400-year flood control dam, they would soon exceed the critical phreatic lag value and would probably mobilize.

During preliminary engineering and design, all factors that affect soil stability in the inundation zone will need to be evaluated more closely to properly size the outlet to achieve an optimum drawdown rate to preclude widespread soil instability within the reservoir.

Figure 15
SUMMARY OF 50 PERCENT SLOPE MODEL RESULTS

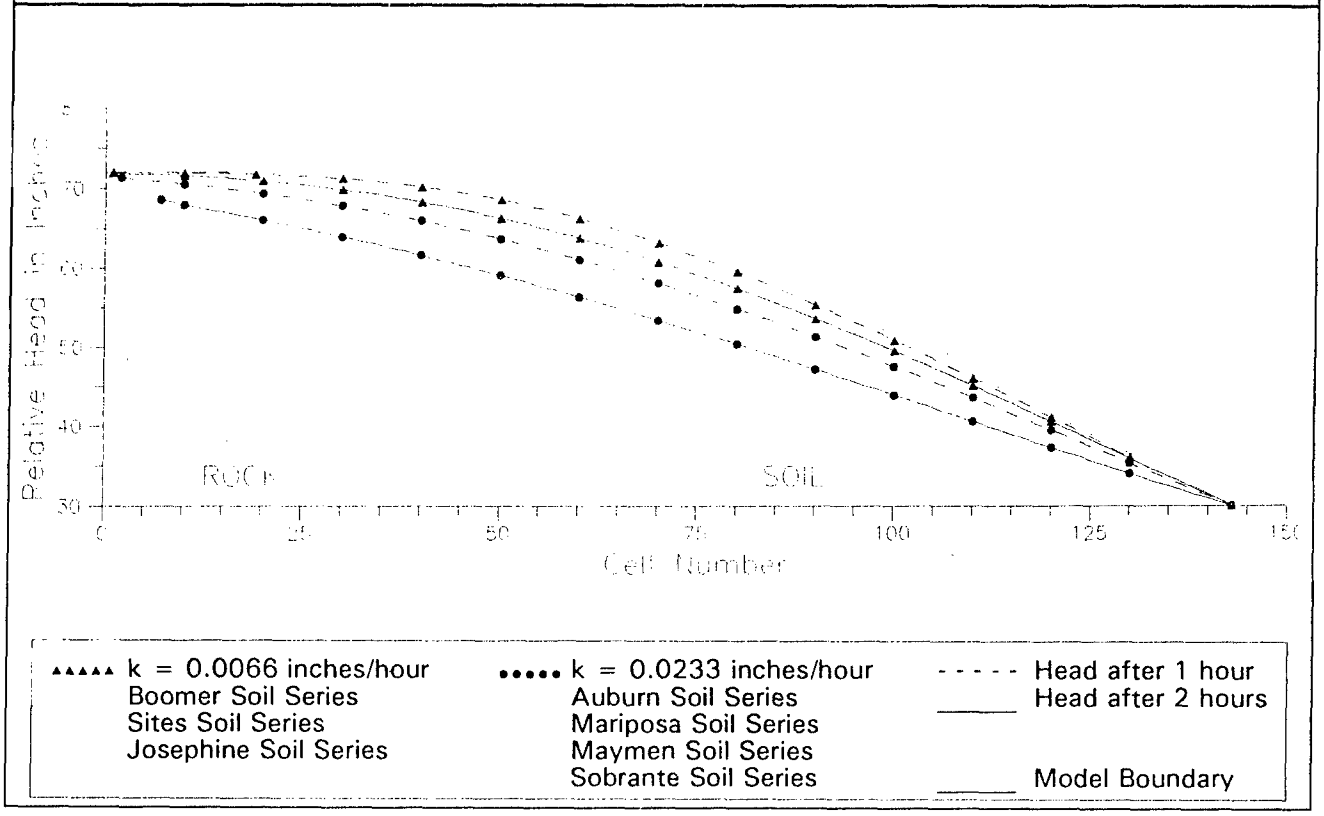
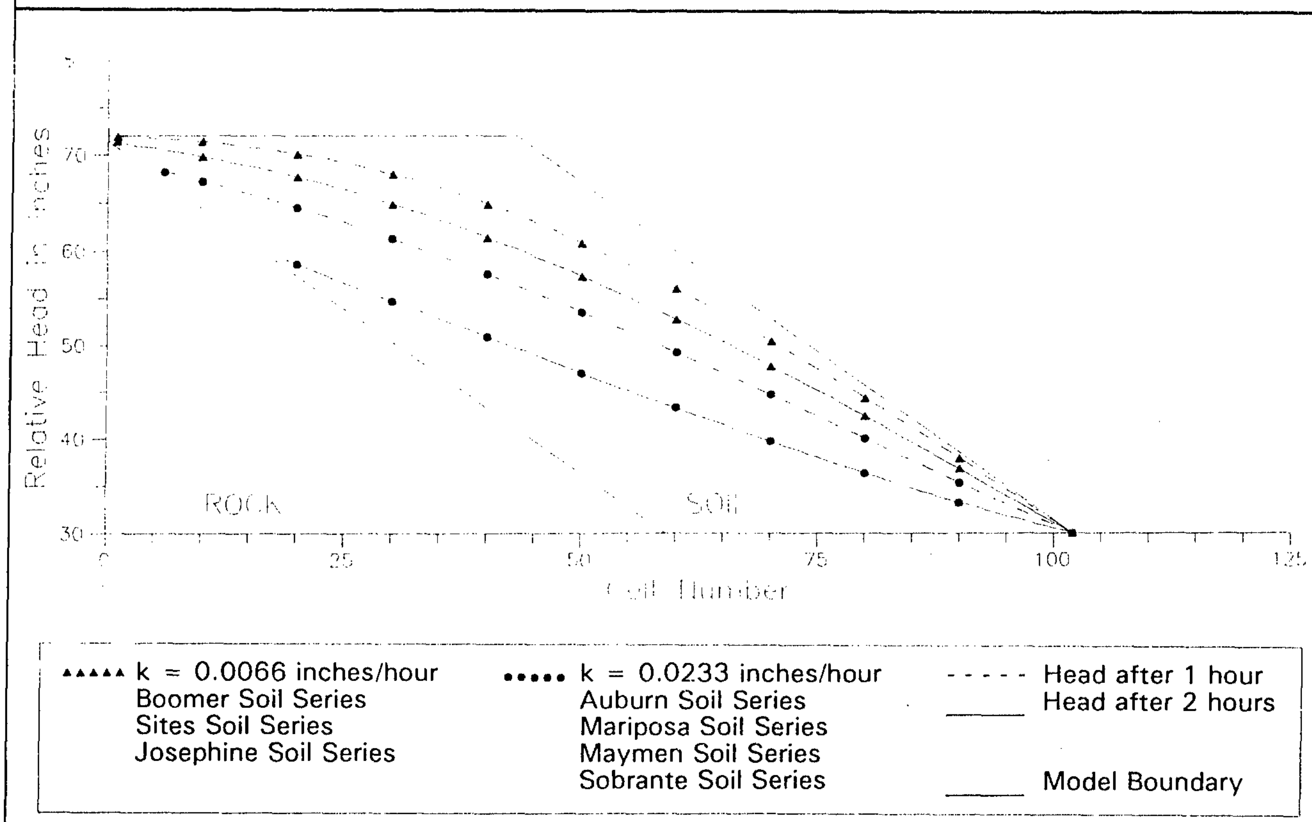


Figure 16
SUMMARY OF 70 PERCENT SLOPE MODEL RESULTS



FIELD OBSERVATIONS

Field observation in the Auburn cofferdam reservoir area provided some confirmation of model results.

In the upper portions of the reservoir, where effects of the very rapid drawdown were probably at a minimum, nearly all the higher permeability Auburn series soils had remained stable following reservoir inundation and drawdown. Where slope failures were seen in Auburn soils they could be attributed to undercutting the toe of the soil mass.

The lower permeability Boomer soils examined did experience some distress as a result of reservoir inundation and rapid lowering. Near the high water level, which was marked by a driftwood line, prominent scarps developed as the soil and underlying regolith (older landslide material) moved in a rotational manner downslope. The estimated maximum movement was less than 50 feet. At this point it is unclear whether the movement occurred as a result of reactivation of the older landslide mass or failure of the Boomer soils that develop upon it. Further exploration would be needed to clarify this issue. Although the Boomer soils and underlying regolith showed a tendency to move, it is important to remember that they move on their own when saturated by prolonged heavy rainfall. Many trees on the Boomer soils had curved trunks, indicating movement of the soil mass had occurred prior to inundation by the cofferdam reservoir and sometime after the trees had become established. Ironically, conditions that would impound water behind the flood control dam are the same conditions that would probably cause the Boomer and similar low permeability soils to move on their own.

The Fish and Wildlife Service report concluded that wavewash would contribute to removal of soils from the reservoir area. Inspection of soils near the 1986 driftwood line found no evidence of soil removal by wavewash. Although there is some potential for wavewash to occur, the reservoir should not be held at any one elevation long enough to allow any significant soil removal. Moreover, the natural vegetative cover should provide some limited riprap effect to repress infrequent wavewash erosion. Soil formation and soil downslope migration in the reservoir are dynamic processes. Soil that may be removed by the very infrequent inundation probably would be replaced over time by natural soil migration and formation processes.

CONCLUSIONS AND RECOMMENDATION

Following are conclusions resulting from these evaluations of soils and soil stability for the proposed flood control dam at Auburn.

- The 10 soil series in the area of the proposed flood control dam are: Auburn, Boomer, Exchequer, Horseshoe, Inks, Josephine, Mariposa, Maymen, Sites, and Sobrante.
- Of these 10 soil series, 7 are found in the inundation zone for the proposed 200- and 400-year flood control dam: Auburn, Boomer, Josephine, Mariposa, Maymen, Sites, and Sobrante.
- The Auburn, Sites, Sobrante, and Exchequer soil series are fine-grained soils. The Mariposa, Maymen, and Inks soil series are coarse-grained soils. The Boomer, Josephine, and Horseshoe soil series very from fine-grained to coarse-grained soil. All coarse-grained soils in the project area contain a high percentage of fine-grained soil.
- Permeability of all the soils in the inundation zone for the proposed 200- and 400-year flood control dam fall into two ranges: a relatively higher permeability range of 0.033 to 0.010 inches per minute for the Auburn, Mariposa, Maymen, and Sobrante soil series; and a lower permeability range of 0.010 to 0.0033 inches per minute for the Boomer, Sites, and Josephine soil series.
- Results of a computer model to simulate drainage of a soil column indicate that the higher permeability soils drain under gravity at a rate ranging from 0.7 foot per hour on 50 percent slopes and 0.8 foot per hour on 70 percent slopes. The simulation also indicated that soils in the lower permeability range drained more slowly, 0.4 foot per hour on 50 percent slopes and 0.5 foot per hour on 70 percent slopes. Additional data and analysis are needed to further analyze the drainage rates of soils in the inundation zone.
- About 35 percent of the reservoir area is rock or riverwash materials that will not be affected by periodic inundation. About 50 percent of the reservoir is veneered with higher permeability soils that should remain stable at drawdown rates specified for a 400-year flood control dam. The stability of these same soils is suspect at drawdown rates specified for a 200-year dam. Stability of the remaining 15 percent of reservoir soils, which are lower permeability, is also suspect at drawdown rates specified for either the 200- or 400-year flood control dam. Additional testing and analysis will be needed to further evaluate the stability of all soils series within the inundation zone.
- Wavewash should not be a serious problem with respect to removal of soils in the inundation zone for the proposed reservoir.

During preliminary engineering and design, field and laboratory testing of each soil series should be done to verify results of the modeling and determine an optimum drawdown rate in the reservoir to maximize soil stability. This analysis should include additional ground water flow modeling coupled with conventional slope stability analyses.

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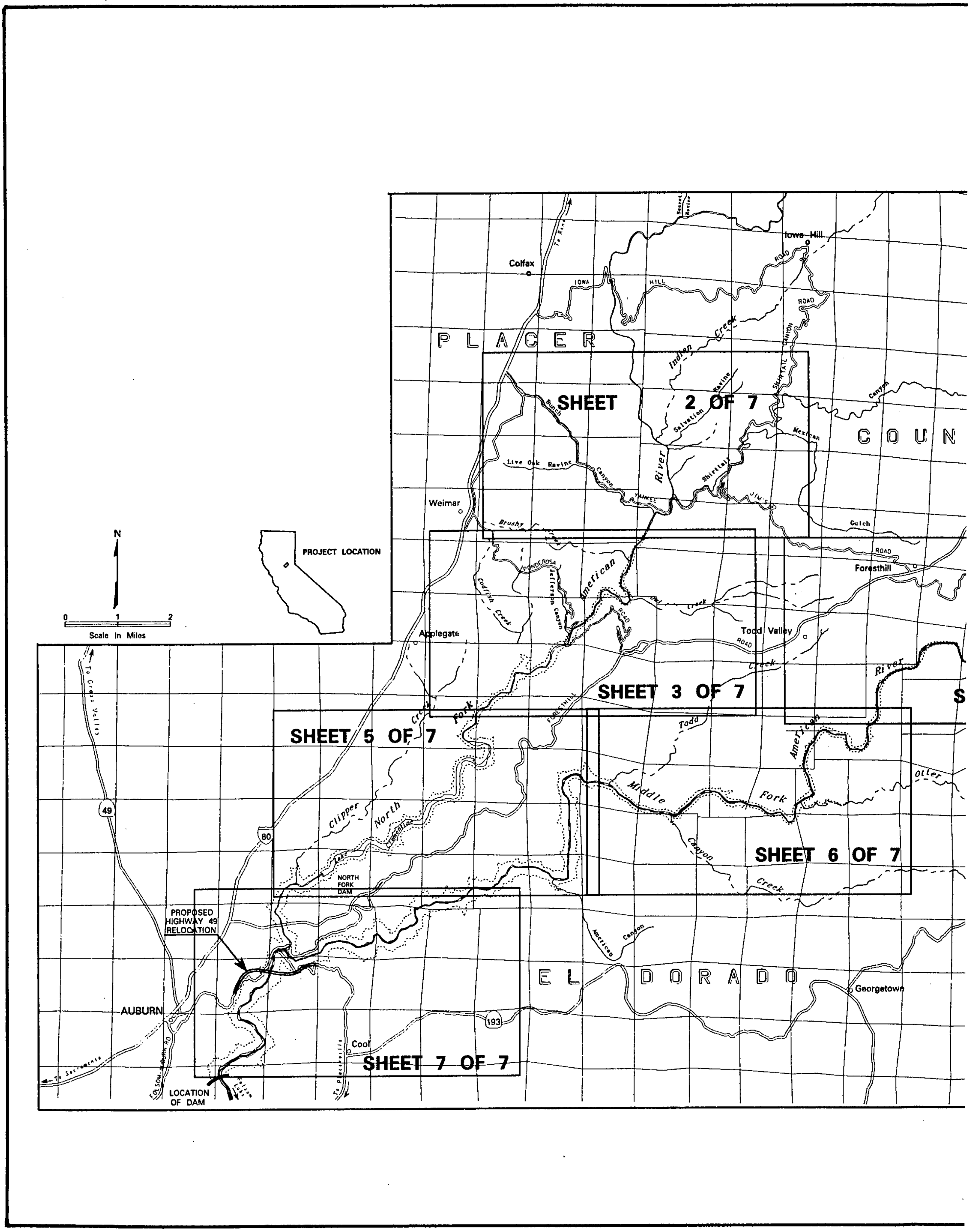
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SHEET 5 OF 7
SHEET 6 OF 7
SHEET 7 OF 7
Colfax
Iowa Hill
Weimar
Applegate
Foresthill
Todd Valley
Auburn
Cool
Georgetown
NORTH FORK DAM
PROPOSED HIGHWAY 49 RELOCATION
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PROJECT LOCATION
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Middle Fork American River
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Shirttail Canyon
Live Oak Ravine
Codfish Creek
Clipper Creek
Todd Creek
Otter Creek
Canyon Creek
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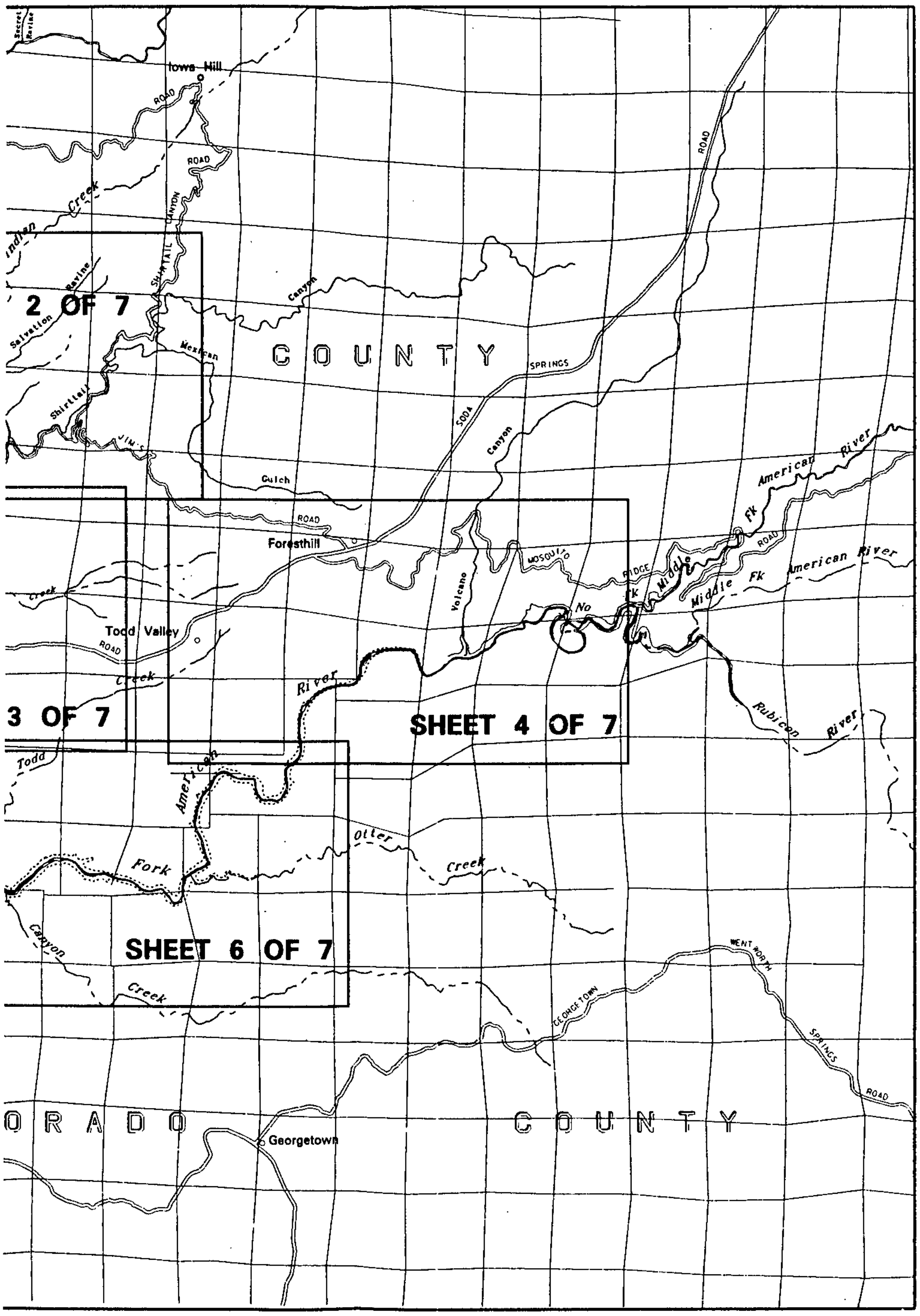
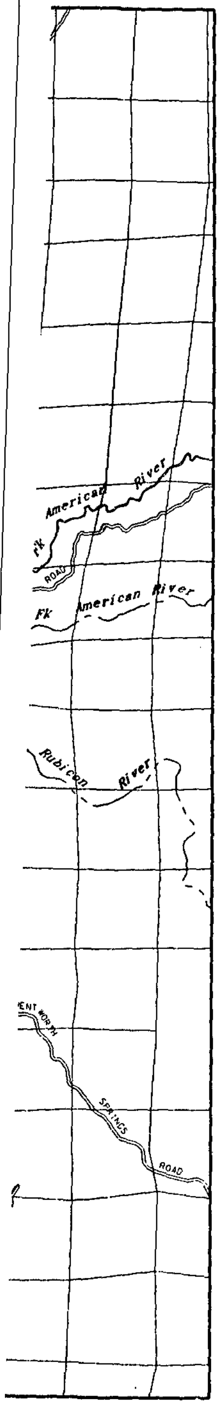
To Grass Valley
To Sacramento
To Placerville



LEGEND

AC	Auburn, silt loam, 2-15% slopes
ARD	Auburn, rock outcrop complex, 2
ARE	Auburn, rock outcrop complex, 2
ARF	Auburn, rock outcrop complex, 2
ASD	Auburn, Sobrante silt loams, 15-
ASRD	Auburn, Sobrante, rock outcrop
ASRE	Auburn, Sobrante, rock outcrop
ASRF	Auburn, Sobrante, rock outcrop
BA	Boomer loam, 2-5% slopes
BD	Boomer loam, 15-30% slopes
BRD	Boomer, rock outcrop complex,
BRE	Boomer, rock outcrop complex,
BRF	Boomer, rock outcrop complex,
HRD	Horseshoe, rubble land complex
HRE	Horseshoe, rubble land complex
IE	Inks, cobbly loam, 30-50% slope
IED	Inks, Exchequer complex, 2-25%
MD	Mariposa, gravelly loam, 5-30%
MJD	Mariposa, Josephine complex, 5
MJE	Mariposa, Josephine complex, 3
MRE	Mariposa, rock outcrop complex
MRF	Mariposa, rock outcrop complex
MYRF	Maymen, rock outcrop co
SRE	Sites, rock outcrop complex, 15-
Rl	Rock land
R	Riverwash
	Quarry and Rubble land



LEGEND

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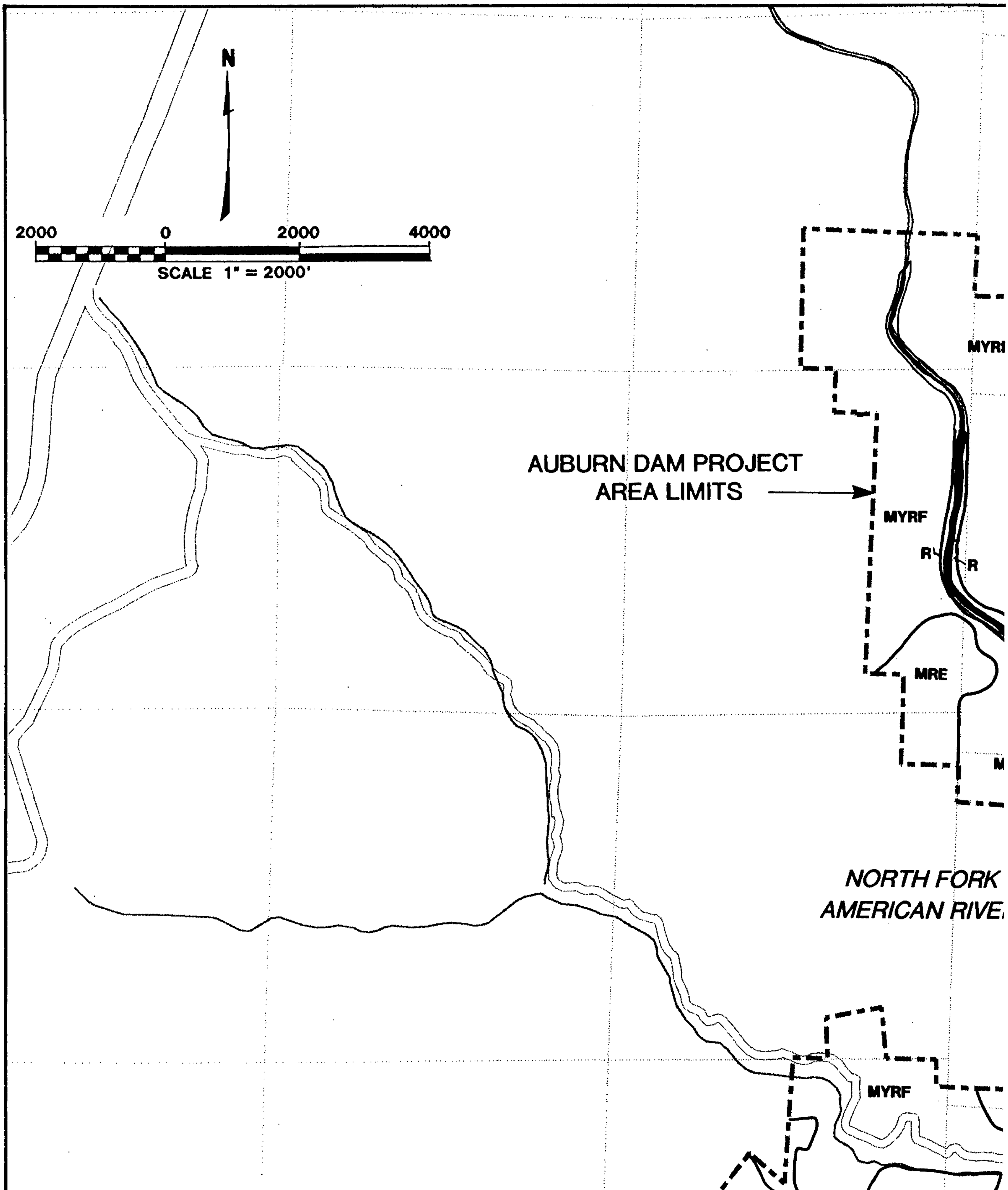
ARD	Auburn, rock outcrop complex, 2-30% slopes
ARE	Auburn, rock outcrop complex, 2-50% slopes
ARF	Auburn, rock outcrop complex, 2-70% slopes
ASD	Auburn, Sobrante silt loams, 15-30% slopes
ASRD	Auburn, Sobrante, rock outcrop complex, 2-30% slopes
ASRE	Auburn, Sobrante, rock outcrop complex, 30-50% slopes
ASRF	Auburn, Sobrante, rock outcrop complex, 50-70% slopes
BA	Boomer loam, 2-5% slopes
BD	Boomer loam, 15-30% slopes
BRD	Boomer, rock outcrop complex, 5-30% slopes
BRE	Boomer, rock outcrop complex, 30-50% slopes
BRF	Boomer, rock outcrop complex, 50-70% slopes
HRD	Horseshoe, rubble land complex, 2-30% slopes
HRE	Horseshoe, rubble land complex, 30-50% slopes
IE	Inks, cobbly loam, 30-50% slopes
IED	Inks, Exchequer complex, 2-25% slopes
MD	Mariposa, gravelly loam, 5-30% slopes
MJD	Mariposa, Josephine complex, 5-30% slopes
MJE	Mariposa, Josephine complex, 30-50% slopes
MRE	Mariposa, rock outcrop complex, 5-50% slopes
MRF	Mariposa, rock outcrop complex, 50-70% slopes
MYRF	Maymen, rock outcrop complex, 50-75% slopes
SRE	Sites, rock outcrop complex, 15-50% slopes
Rl	Rock land
R	Riverwash
	Quarry and Rubble land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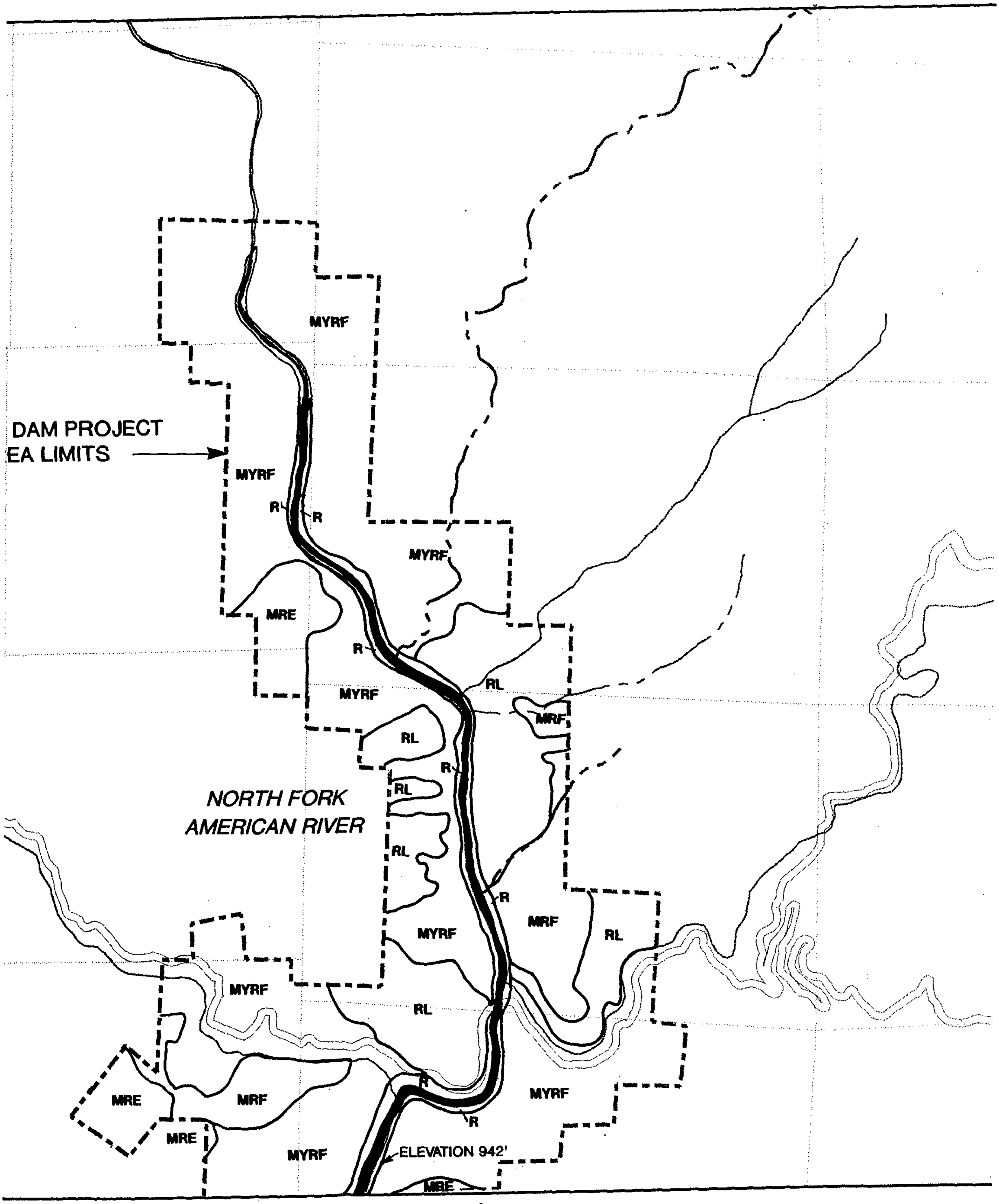
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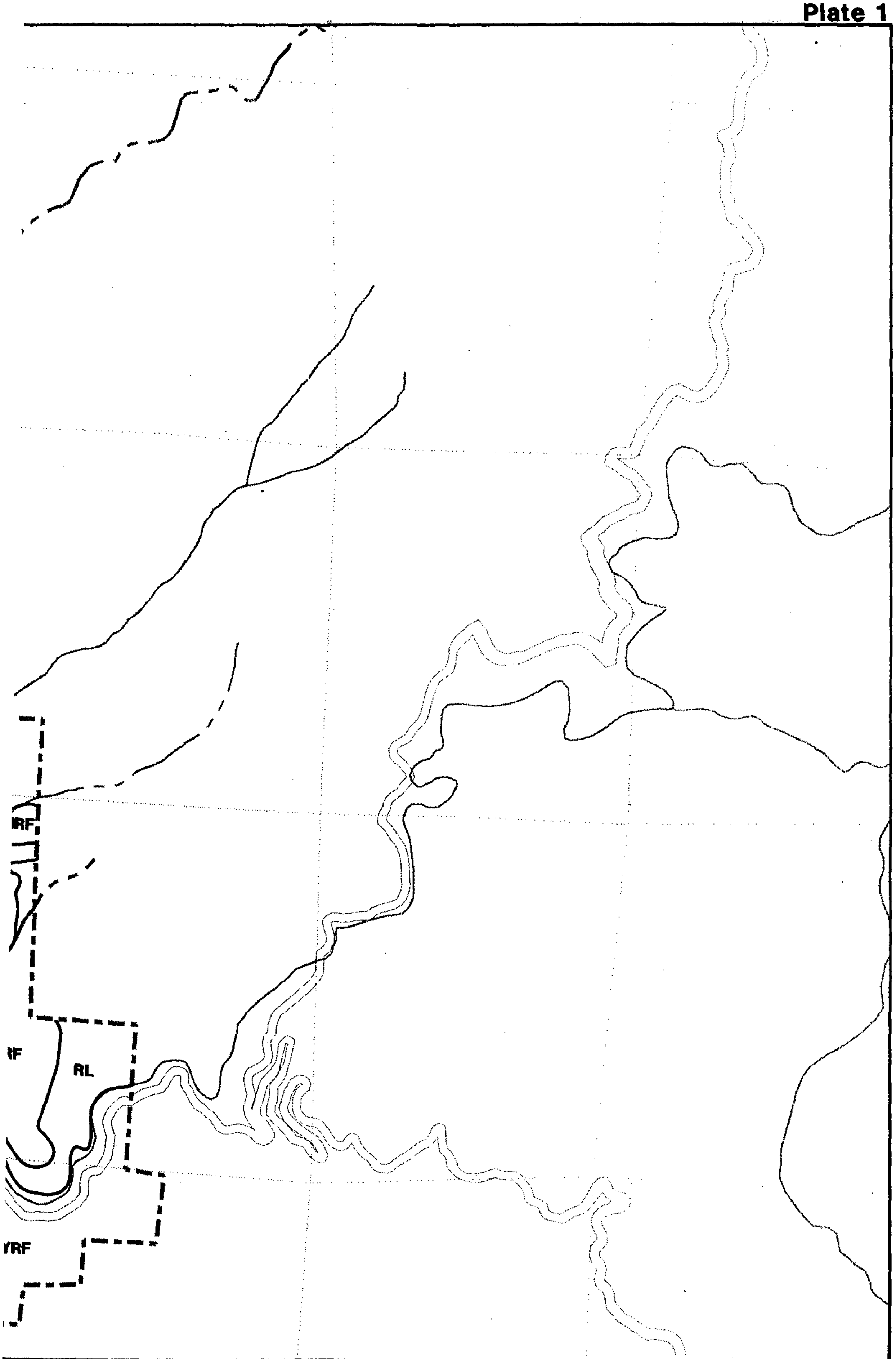

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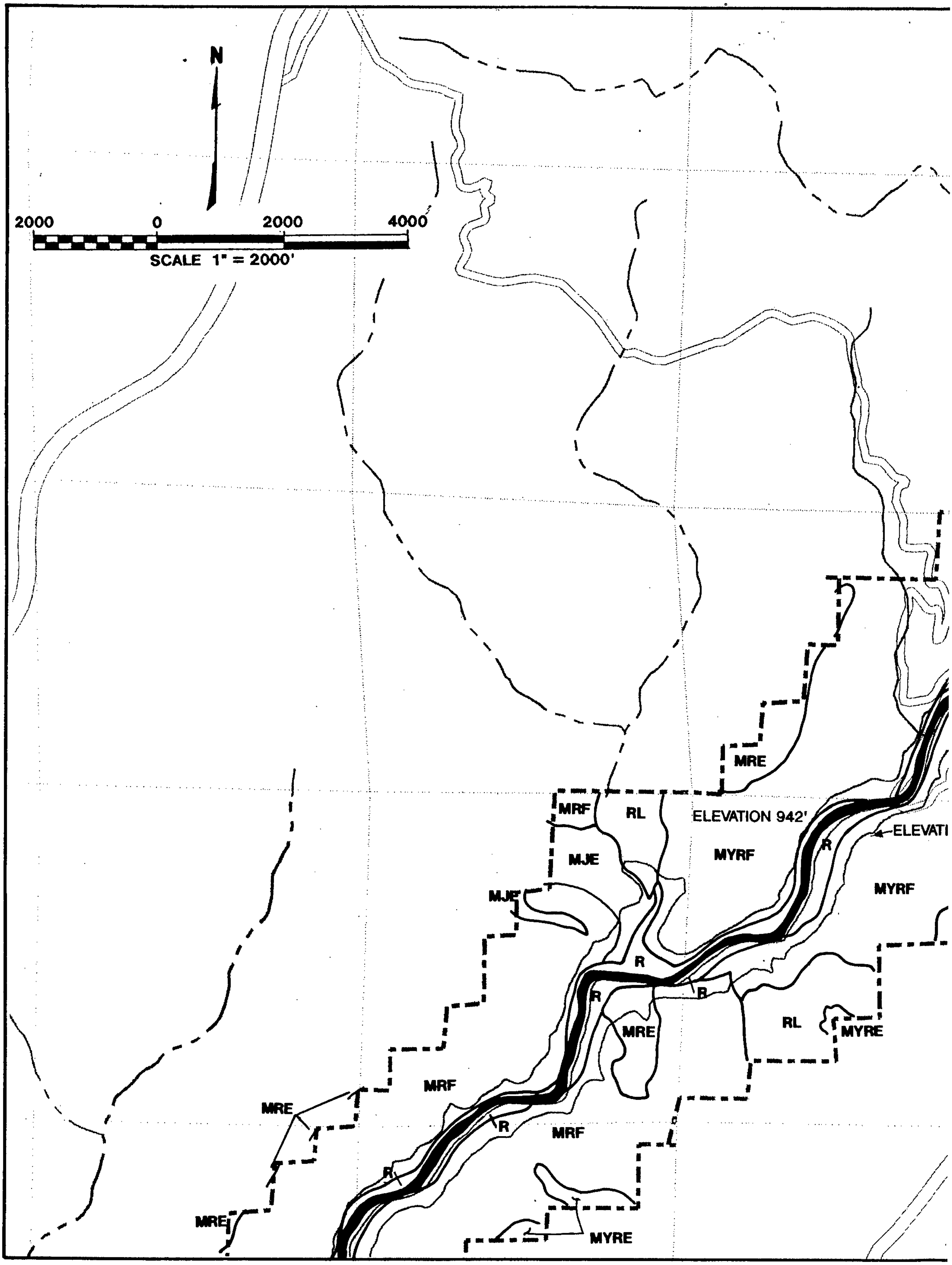

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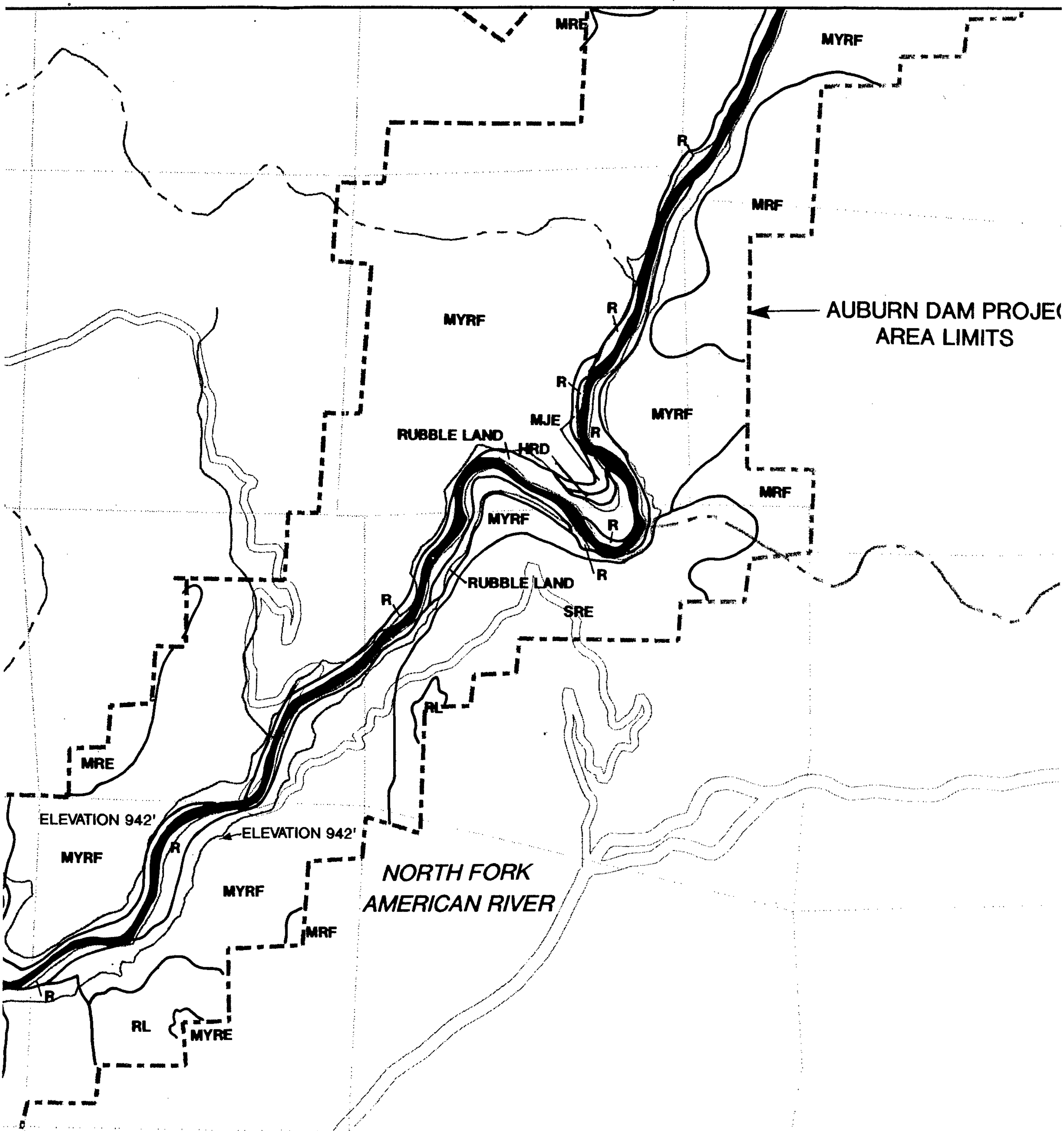
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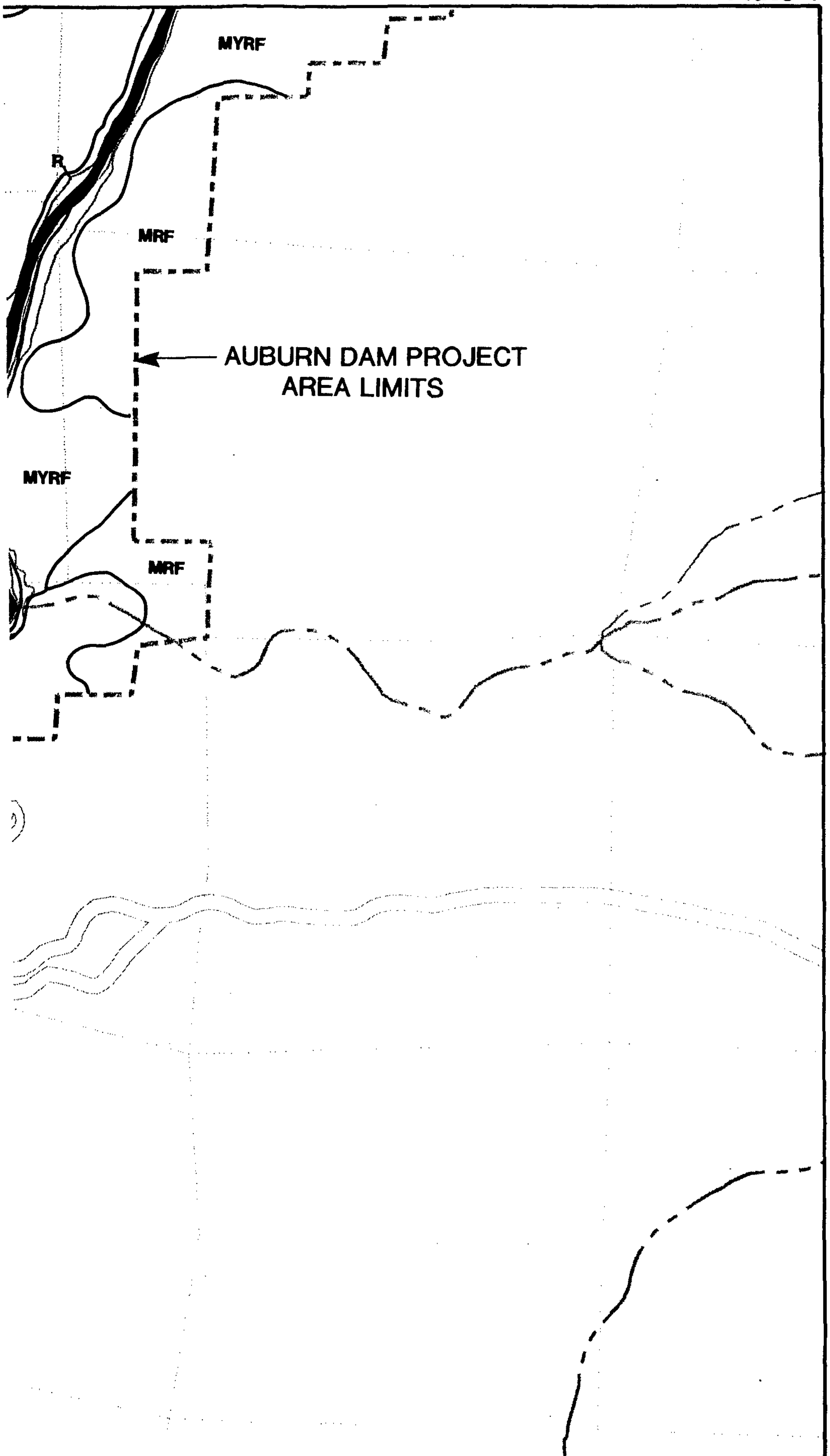
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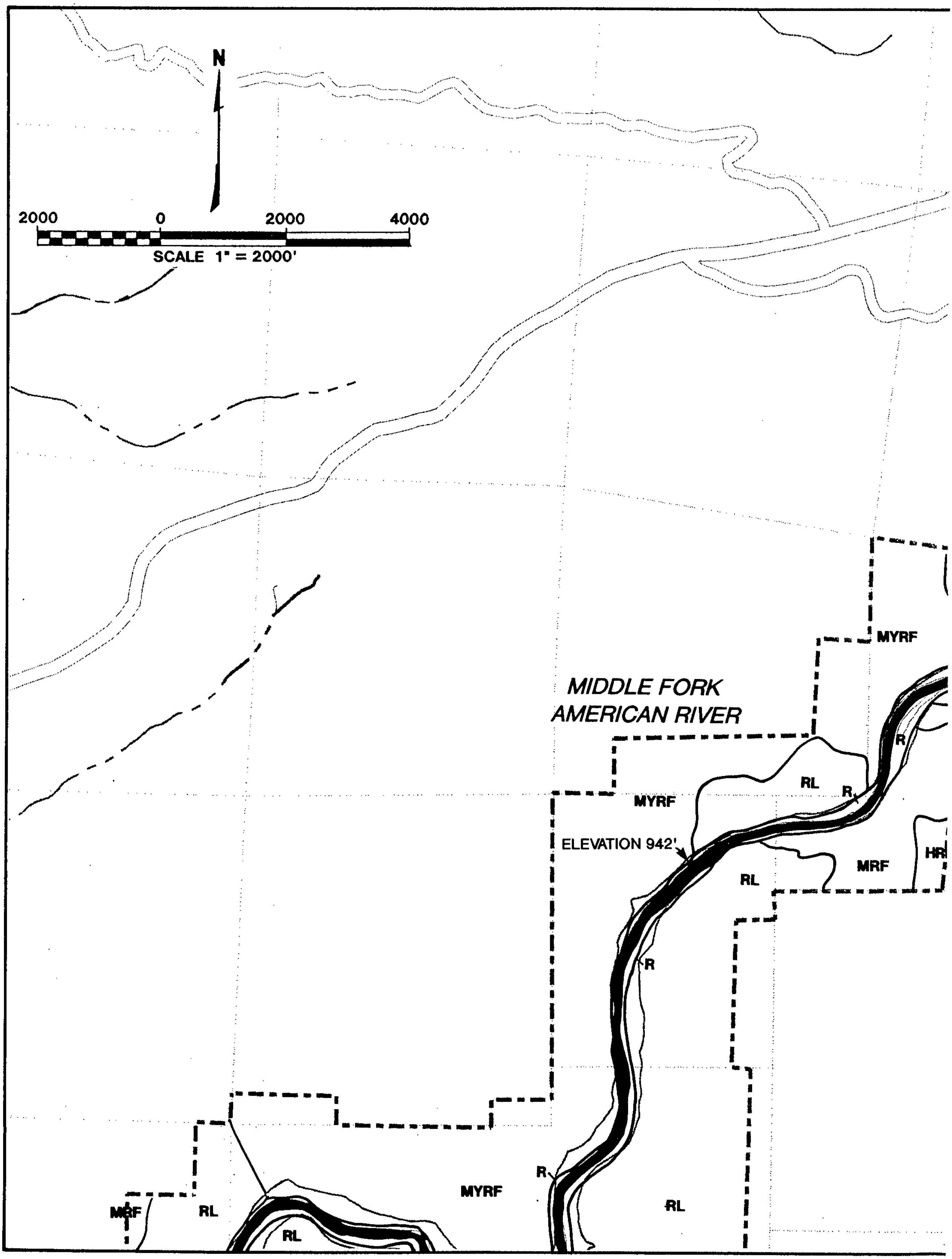

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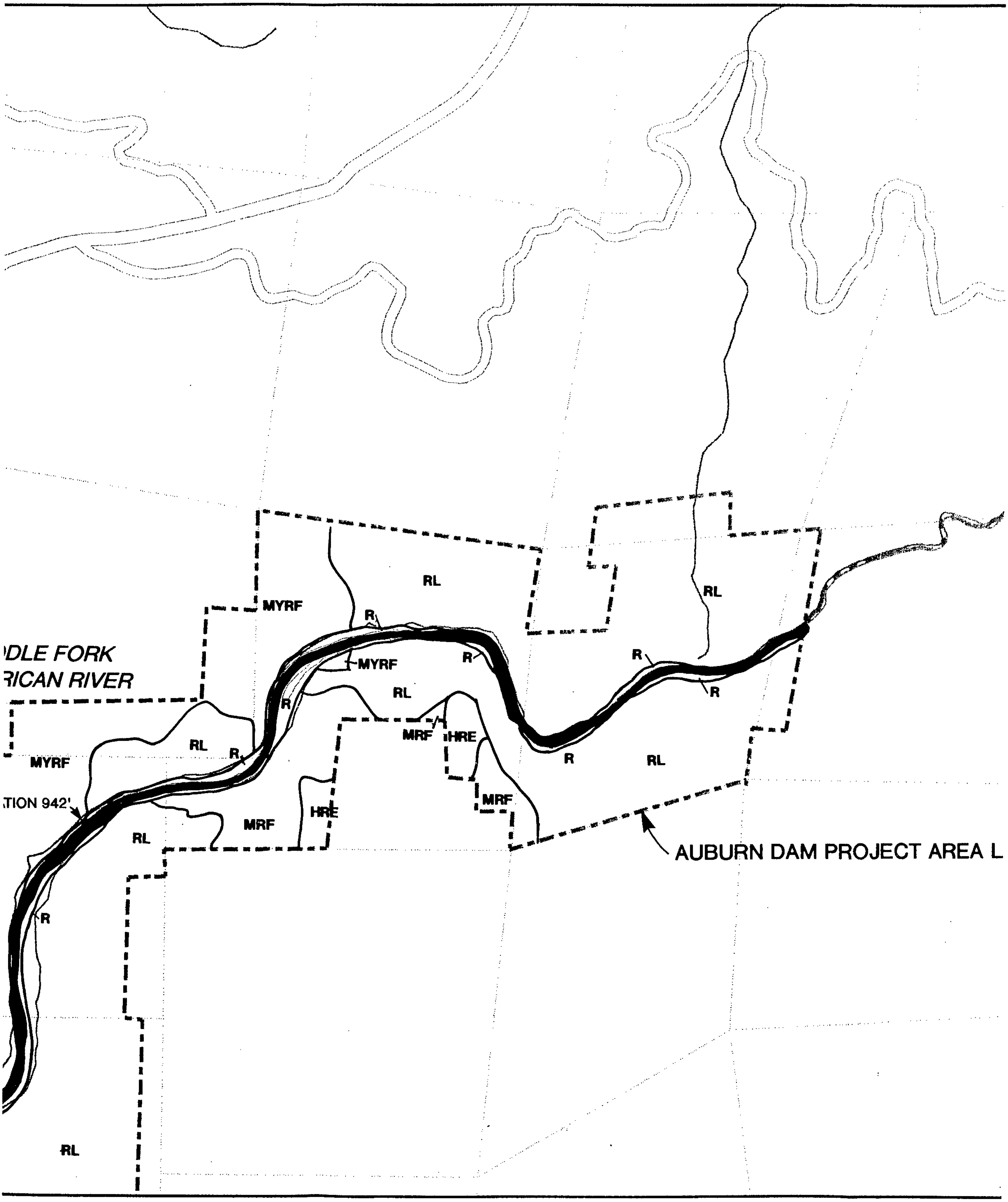
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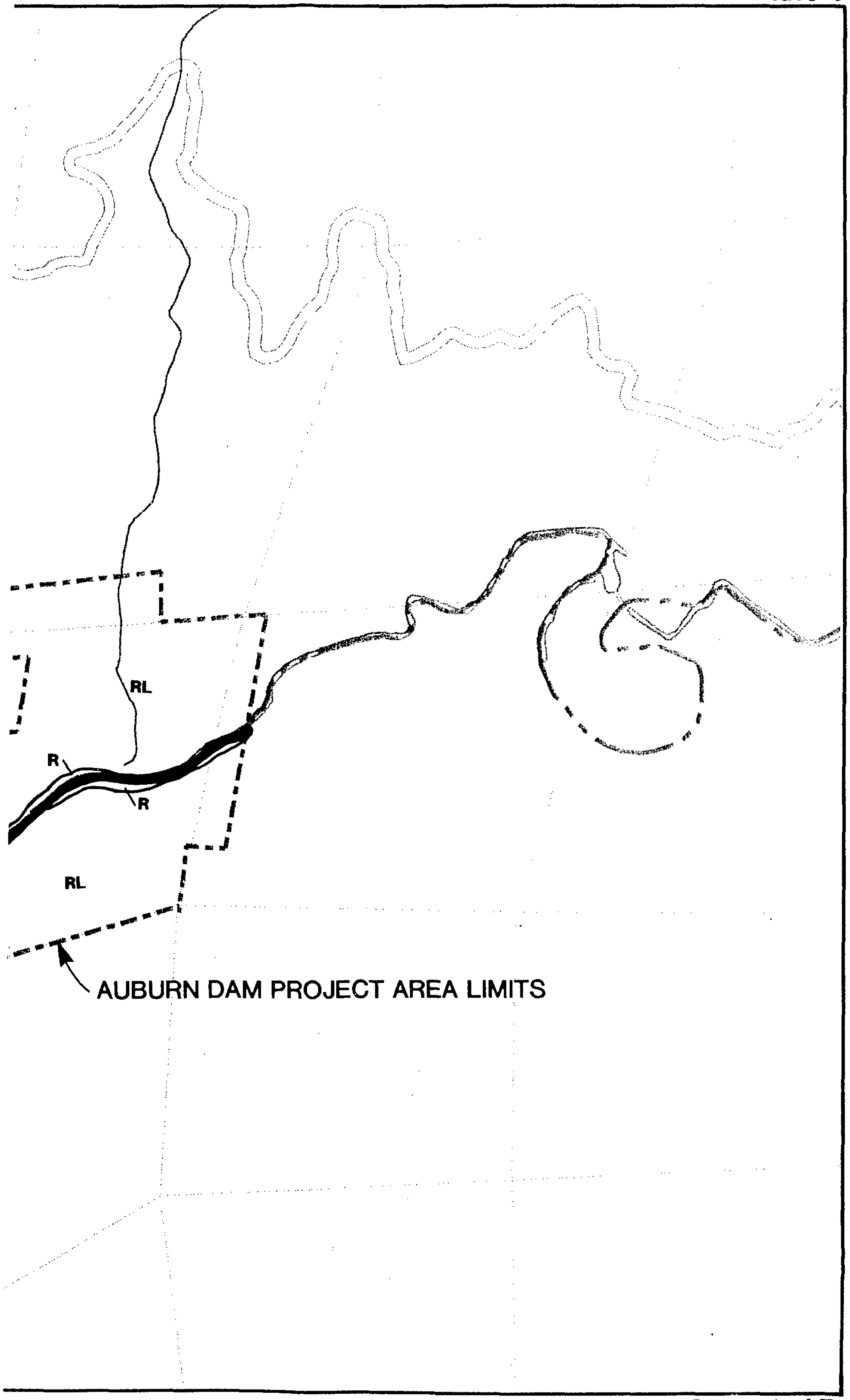
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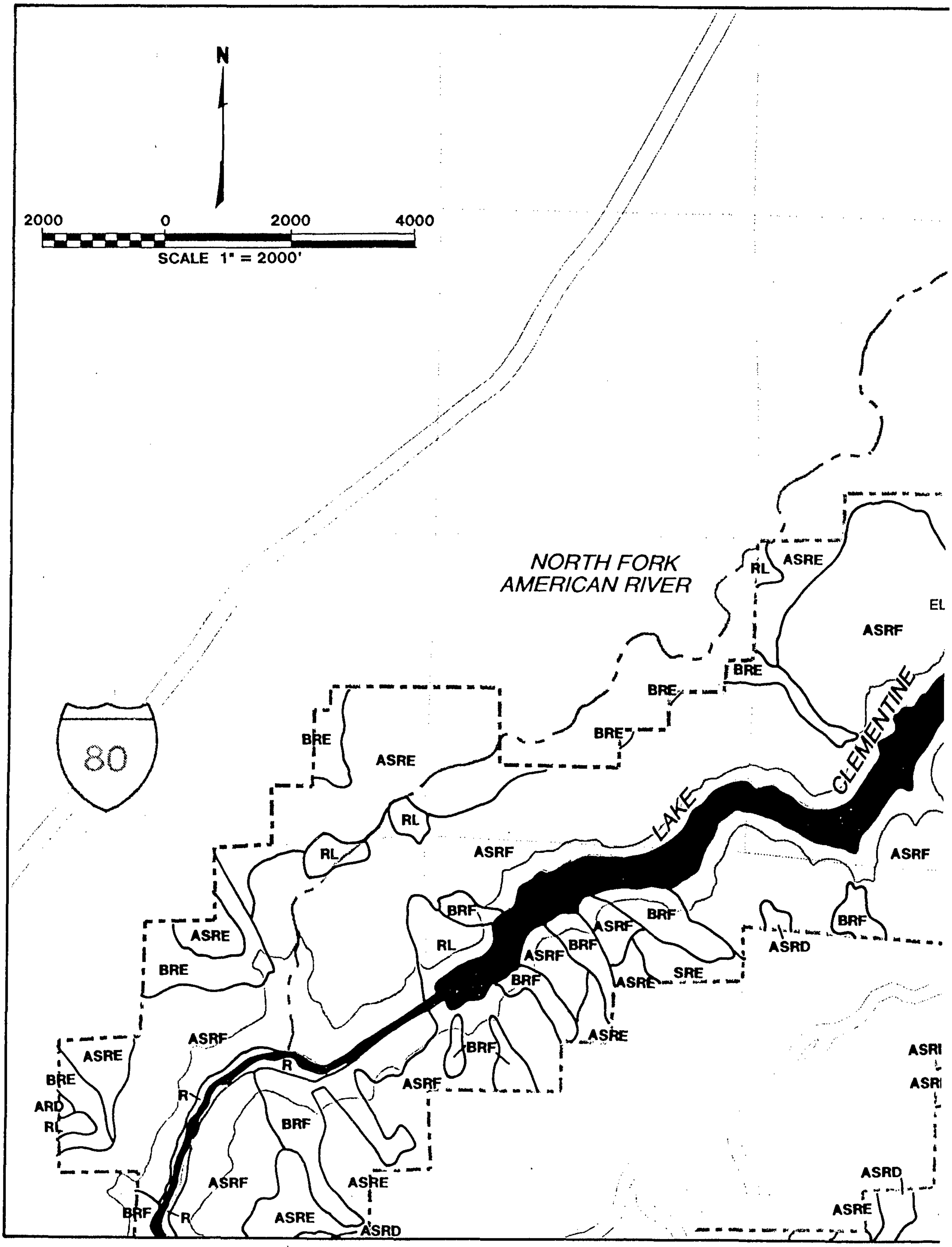

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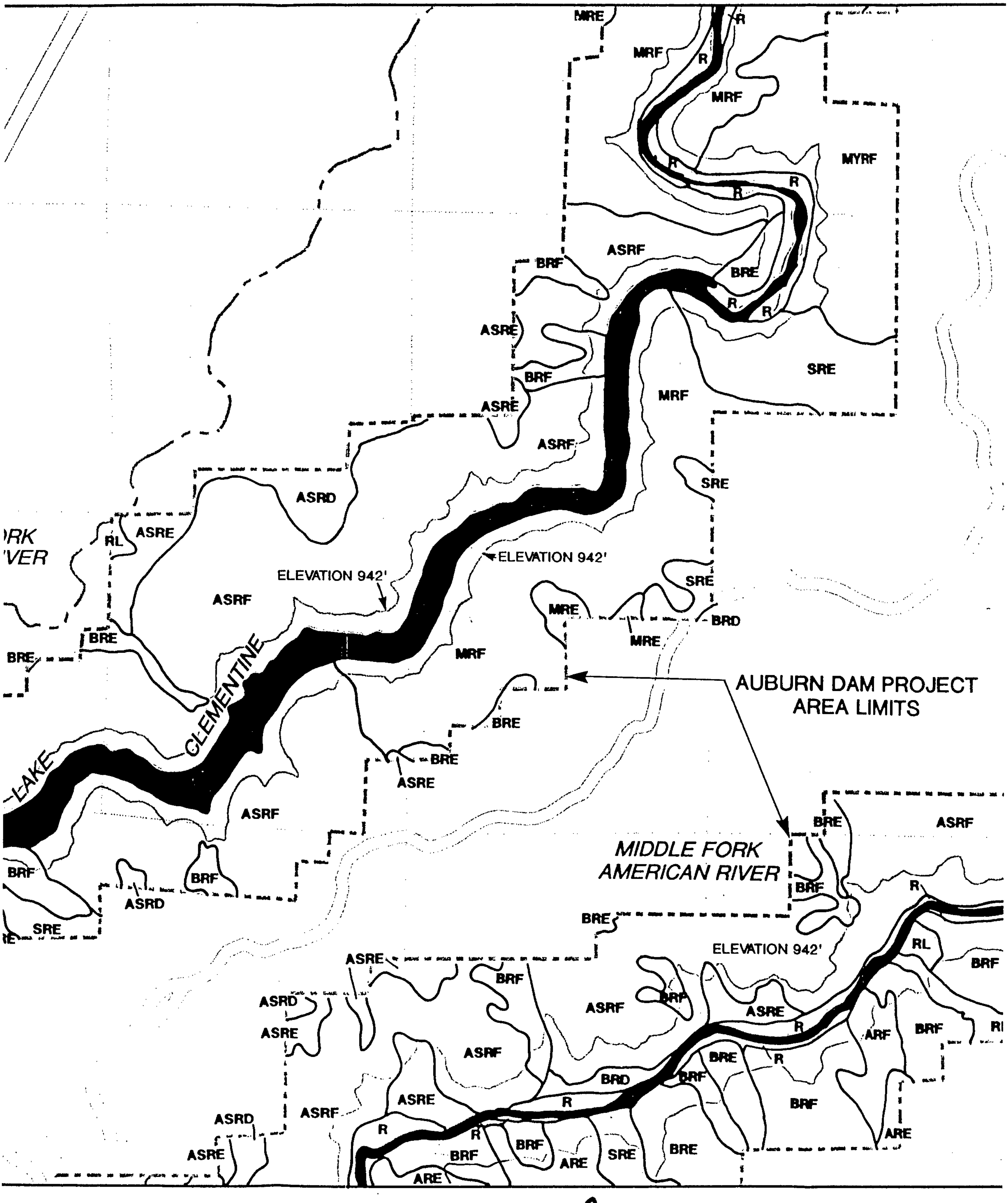
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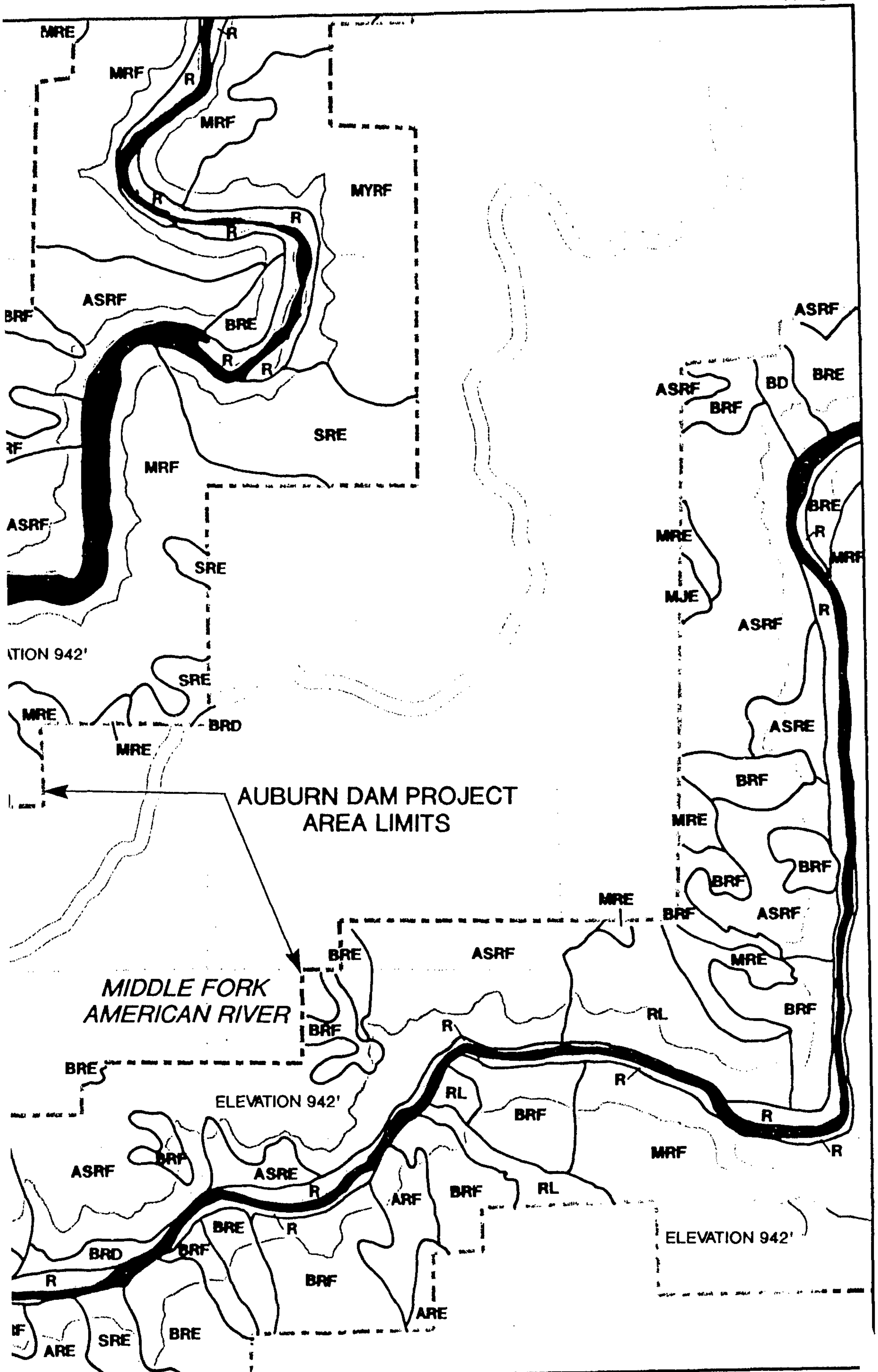
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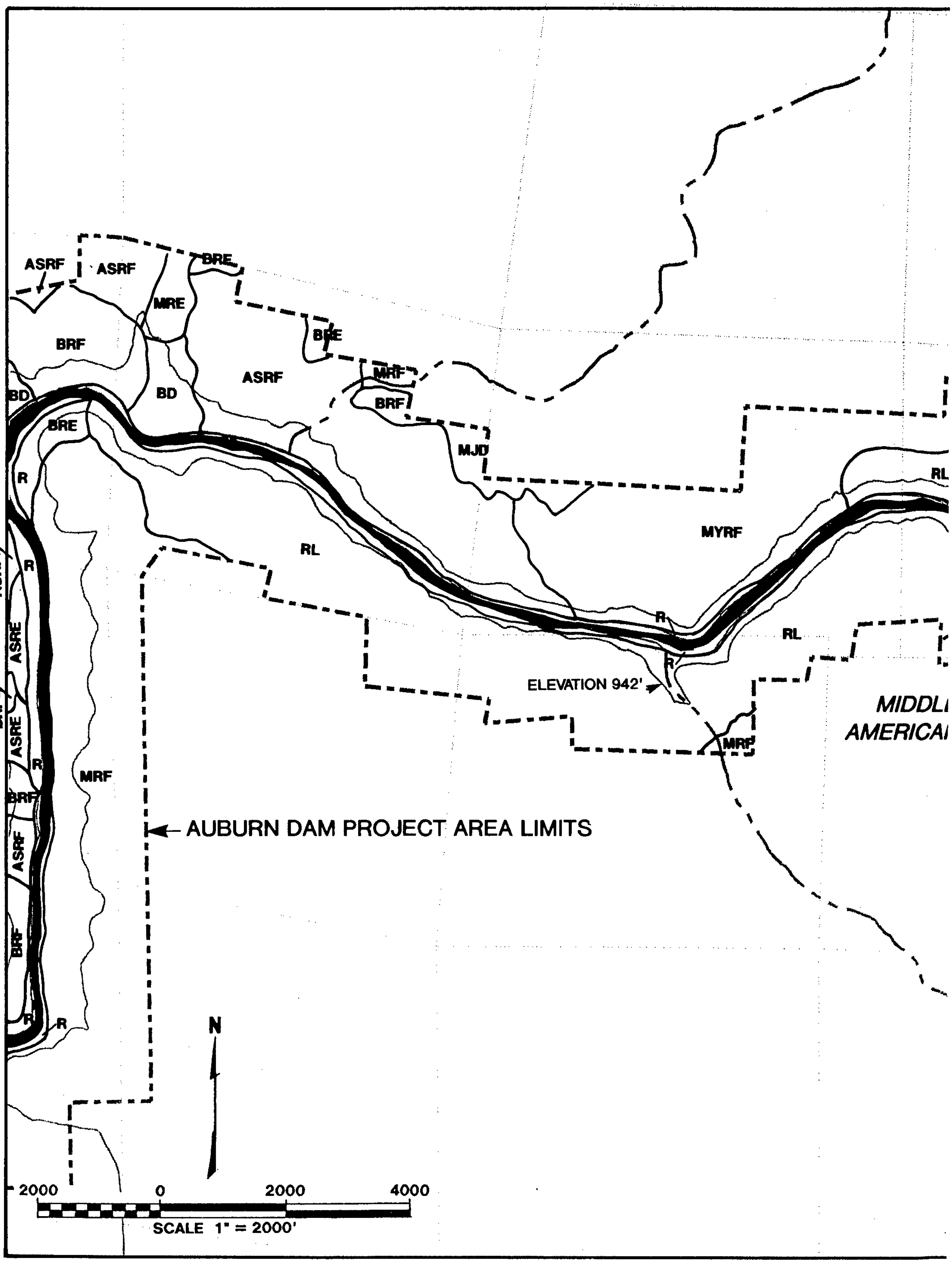
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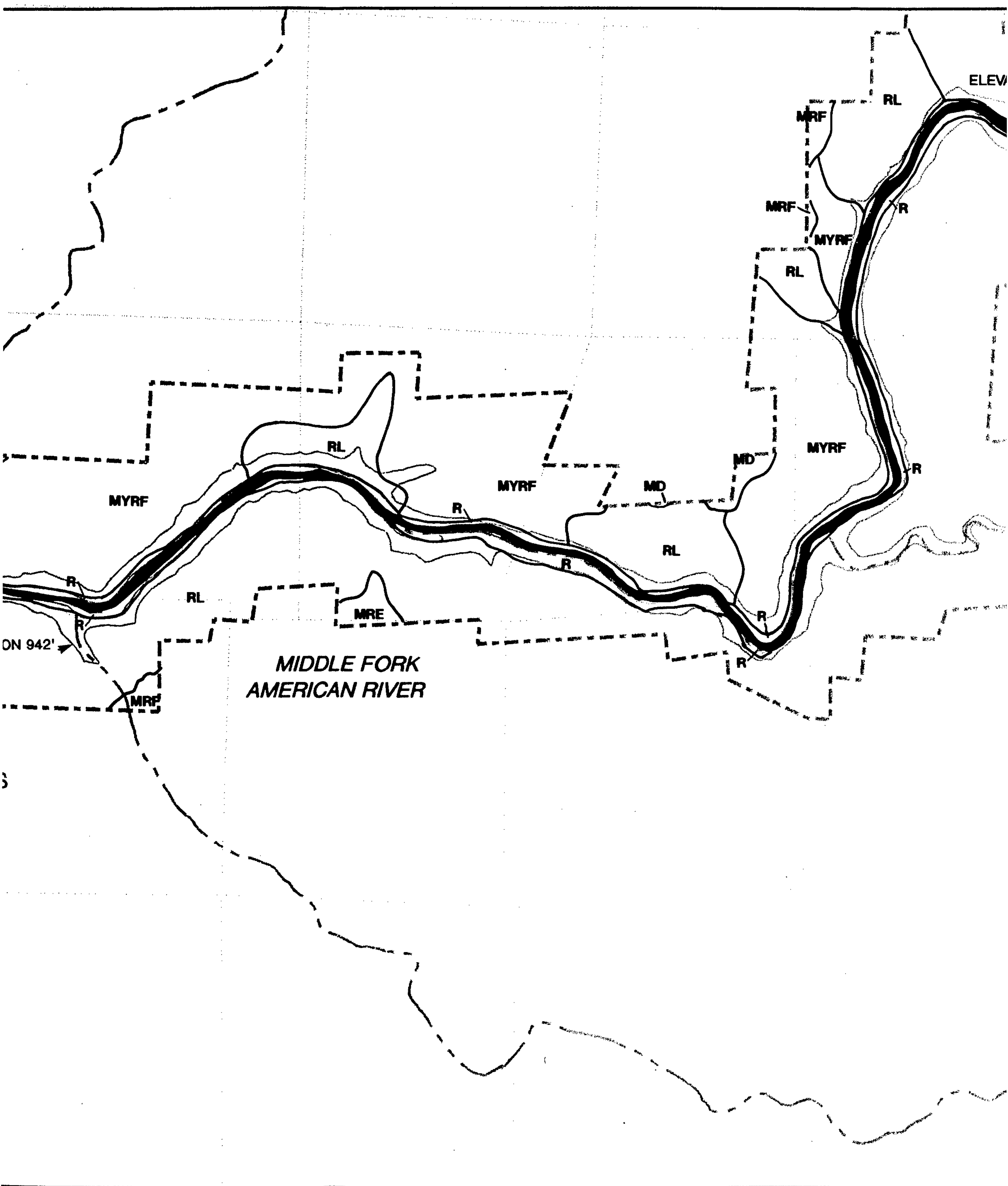
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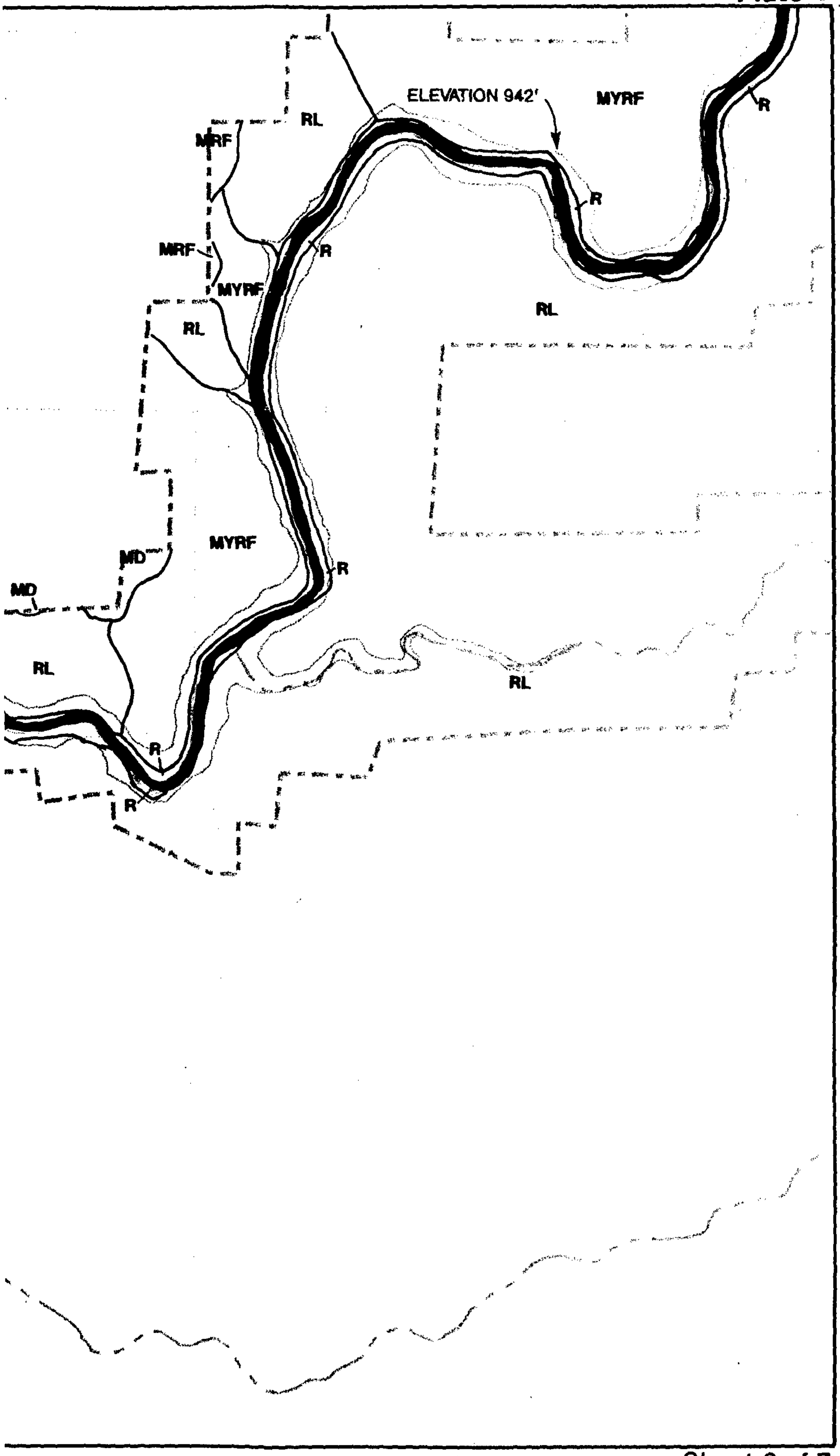
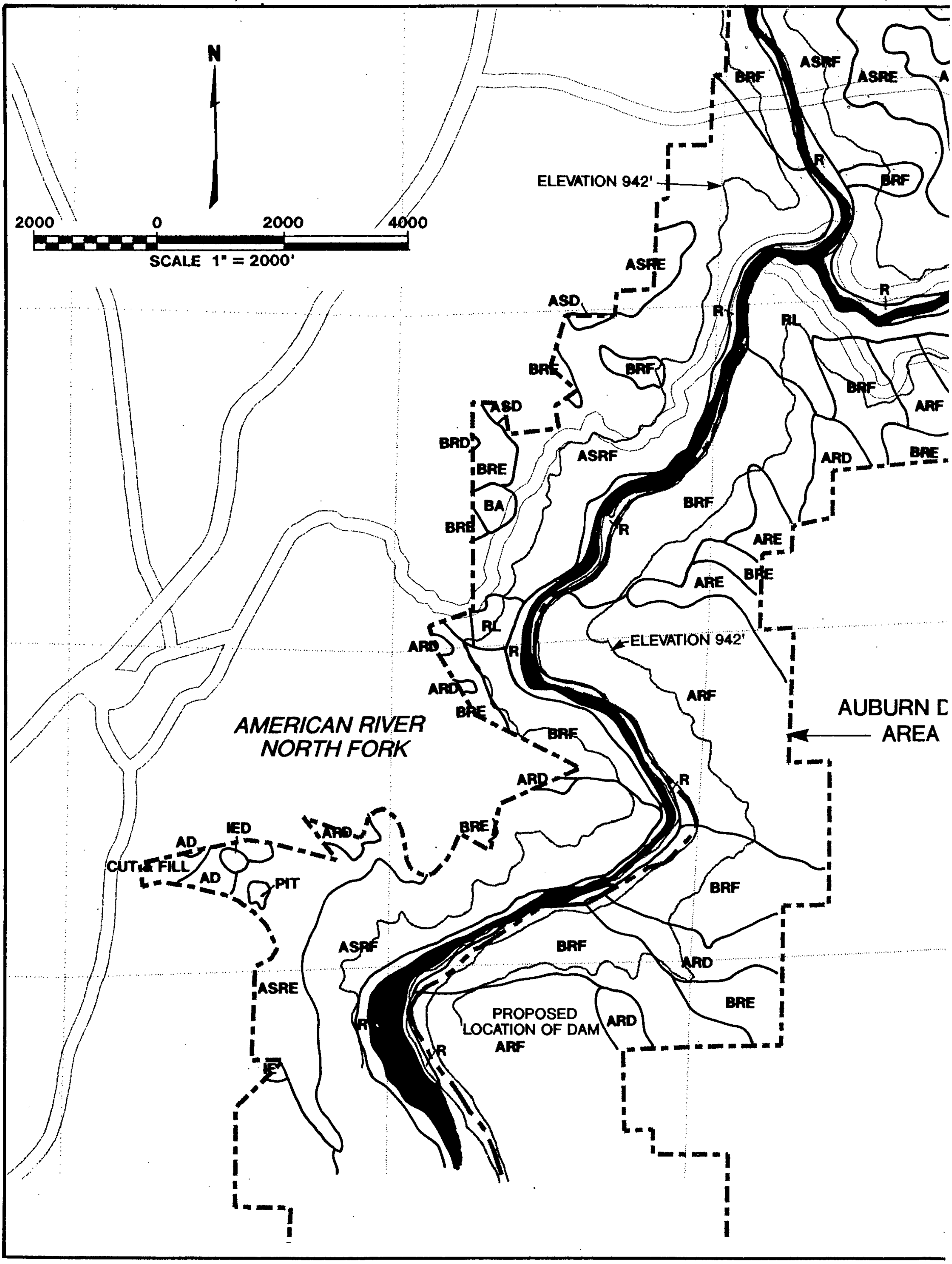
Plate 1



Sheet 6 of 7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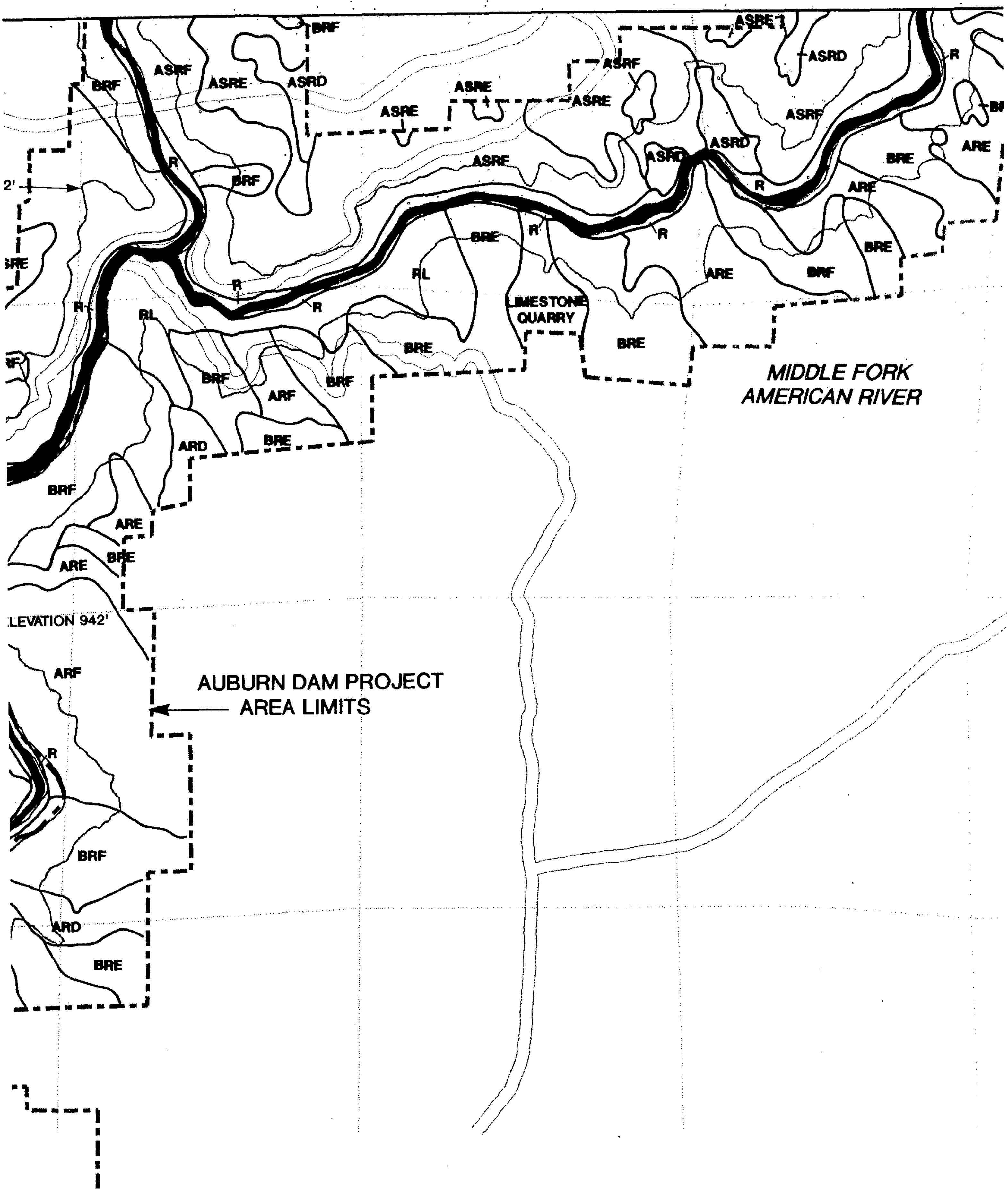
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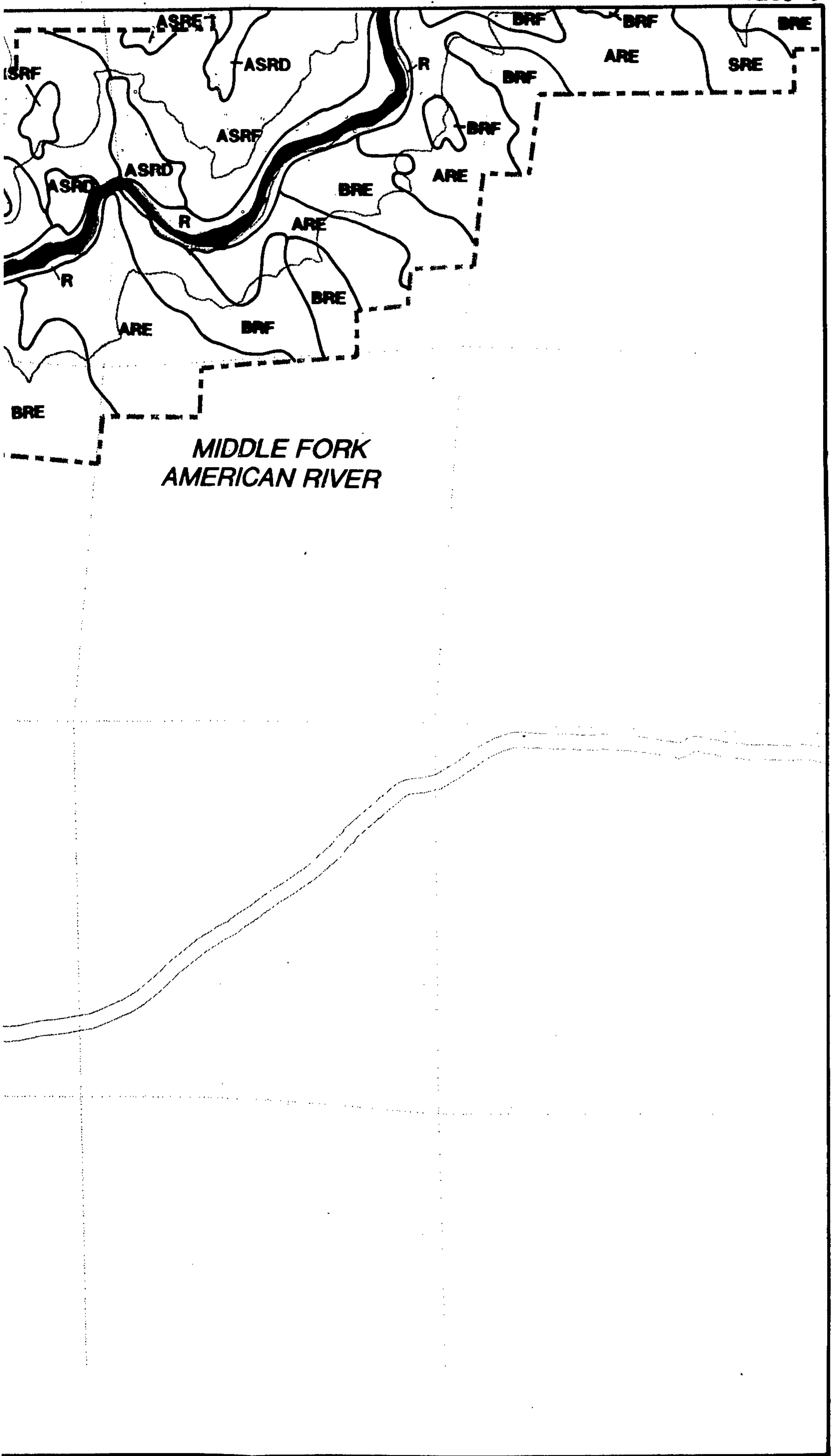
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AMERICAN RIVER

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 9

SPECIAL AGGREGATE STUDY
JUNE 1991

US ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

FLOOD CONTROL DAM
AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

SPECIAL AGGREGATE STUDY

Table of Contents

Figures

Photographs

FLOOD CONTROL DAM
AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

SPECIAL AGGREGATE STUDY

1. Purpose and Scope. - This special aggregate study was prepared to answer questions related to extraction of aggregates from the Middle Fork of the American River for the flood control dam portion of the American River Watershed Investigation. It has been determined that the impacts of aggregate extraction from the Middle Fork of the American River requires expanded description in the EIS. In order to address these impacts, a list of questions were compiled for response. Those questions were as follows:

RIVER BAR SOURCE
Aerial extent and depths of extraction at each bar site
Description of excavation equipment, methods and procedures
Location of non-useable material stockpiles at the bars
Location and aerial extent of access roads to each bar

PROCESSING
Location and aerial extent of transport and processing facilities
Description of processing facilities and operations
Location of sand source

ALTERNATE SOURCES
Information on why Lake Clementine is not a feasible source
Aerial extent and potential sites of quarry source
Potential access routes to a quarry
Possible methods of quarry remediation
Possible methods of extraction and transport
Information on other aggregate sources and discussion of material suitability of these sources

This report comprises the response to these questions. The scope of this report is limited to the questions developed. Information for this report was obtained from the files and reports compiled by the Bureau of Reclamation (USBR) from 1967 to 1981, as part of their aggregate studies for a concrete dam at Auburn, from discussion with a variety of construction industry representatives, and from the Reconnaissance Report, dated January 1989, titled "Concrete Materials and Roller Compacted Concrete Dam Considerations", Chapter 6 of this appendix.

2. Project Background. - The most recent information on concrete aggregates for the dam project is contained in the January 1989 Concrete Materials Reconnaissance Report, Chapter 6. Much additional study remains to be done for this project. The information in this report is based on a 200-year level of protection for the American River requiring a 425-foot-high roller compacted concrete (RCC) dam, containing about 5 million cubic yards of concrete. Approximately 6,760,000 cubic yards of aggregate (or 9,125,000 tons) will be required to produce 5,000,000 cubic yards of RCC.

3. <u>Middle Fork American River Bars.</u> - Described in this paragraph is information on the Middle Fork American River bars, quantities of aggregate in each bar, the extent of planned excavation, methods and procedures for excavation, disposal of waste material, control of water turbidity and access to the bars. The Middle Fork bars are entirely Government owned.

a. **Previous Studies.** - The USBR in the 1960's began investigating the sand and gravel bars along the Middle Fork American River as a potential source of concrete aggregate for a multi-purpose Auburn Dam project. These deposits lie along an approximate seven mile reach of the River starting about five miles upstream of the proposed damsite at Mammoth Bar and ending at Cherokee Bar. The preliminary report on these deposits was published by the USBR in 1967. The scope of the investigation was rather limited and consisted of a compilation of historical data from gold dredger operations along the River during the early to mid-1900's, as well as a few dozer pits in Mammoth and Browns bars. Based on this data, the USBR estimated the total quantity of pit run sand and gravel to be in excess of 8 million cubic yards.

Further studies of these deposits were conducted by the USBR in 1967, with the results published in a 1968 report. The investigations consisted of 35 pits excavated from Mammoth to Poverty Bars with a track mounted dragline shovel equipped with a 2.5 cubic yard bucket, shown in Photo 17 and 18. Additional pits were excavated in Cherokee Bar with a D-8 Dozer. These deposits were described in the report as "...a homogenous mixture of reasonably well-graded clean sand and gravel. The total silt content may range from four to six percent in a greater portion of the bars." The materials are generally free of overburden, but may have localized deposits of silt, wood debris and vegetation. Based on the results of a seismic refraction survey conducted during the study, the depths to bedrock in the bars were generalized as follows: (1) Mammoth and Kennebeck 40-90 feet; (2) Texas, Brown and Poverty 30-40 feet; and (3) Hoosier, Buckeye, Philadelphia and Cherokee 10-20 feet. The seismic velocities through the sand and gravel bars were in the range of 1200-2500 fps, which indicates the deposits are rather loose and unconsolidated. The exposed gravel bars along the Middle Fork were estimated to cover an area in excess of 180 acres, which could yield approximately 6.9 million cubic yards. Materials within the River channel could yield another 1.0 million cubic yards.

Additional mapping and explorations of the bar deposits were done during the period between 1968 and 1970 with the results published in a 1976 addendum to the 1968 report. The explorations consisted of 148 holes which were excavated with a pneumatic clamshell digger, from Mammoth to Cherokee bars. A 38-inch diameter casing was driven as the holes were advanced, which permitted sampling and logging of the deposits down to bedrock. No additional studies of any consequence have been done of the channel deposits along the Middle Fork American River since the completion of the 1976 report by the USBR (confirmed by a conversation with Jim Oliverson who was the chief investigator for the aggregate studies at Auburn and also by a review of the correspondence files at the Denver and Auburn USBR offices).

b. **Description of Bars.** - A few small bars on the Middle Fork are located upstream of the damsite which were not addressed as part of any of the aggregate studies performed by the USBR. The quantity of sand and gravel in these bars is insufficient to warrant their development as aggregate borrow sources. The deposits which are discussed here are those which were explored by the USBR, and include the following bars: Mammoth, Texas, Brown,

Kennebeck, Hoosier, Buckeye, Maine, Sardine, Philadelphia, Poverty, and Cherokee. Figures 4 and 15 summarize quantity information about each bar. Additional figures are included in this report showing the average gradation of each bar deposit derived from the USBR clamshell explorations, and photographs are also provided of each significant bar. A sequential description of each bar from Mammoth Bar upstream to Cherokee Bar follows.

(1) Mammoth Bar. - Mammoth Bar is the first major bar located upstream of the damsite, and lies on the right bank of the Middle Fork. In general, the deposit contains 4 percent material larger than six inches with a maximum particle size of 12 inches. The fines (minus #200 sieve) content of the pit run material averages 9.5 percent. The pitrun gradation classifies as a GP-GM (poorly graded gravel with silt). The sand content of the bar is around 36 percent. Mammoth Bar covers an area of approximately 971,250 square feet. Based on an average depth of 31.4 feet (which was the average depth to bedrock in the clamshell test holes), Mammoth Bar should yield approximately 1,129,500 cubic yards of aggregate.

(2) Texas Bar. - Texas Bar is just upstream of Mammoth Bar and lies along both banks of the Middle Fork. The average gradation is nearly the same as that of Mammoth Bar. Texas Bar covers an area of approximately 996,000 square feet, and has an average depth to bedrock of 31.2 feet. The projected yield of the bar is 1,150,900 cubic yards of aggregate.

(3) Browns Bar. - Browns Bar is located along the left bank of the Middle Fork immediately upstream of Texas Bar. The pitrun material classifies as a GW-GM (well-graded gravel with silt), and contains approximately 36 percent sand and 7 percent fines. The bar covers an area of approximately 599,000 square feet with an average depth to bedrock of 30.4 feet. It is estimated that the bar contains approximately 674,400 cubic yards of aggregate.

(4) Kennebeck Bar. - Most of the material from Kennebeck Bar is located along the right bank of the Middle Fork, just upstream of Browns Bar. The pitrun material classifies as a SP-SM (poorly graded sand with silt), and contains approximately 51 percent sand and 6 percent fines. The bar covers an area of approximately 719,000 square feet with an average depth to bedrock of 31.5 feet. It is estimated that the bar contains approximately 839,000 cubic yards of material.

(5) Hoosier Bar. - Hoosier Bar is located along the right bank of the Middle Fork. The pitrun material classifies as a GP-GM (poorly graded gravel with silt), and contains 40 percent sand and 6 percent fines. The bar covers an area of approximately 649,000 square feet with an average depth to bedrock of 25.9 feet. It is estimated that the bar contains approximately 622,500 cubic yards of material.

(6) Buckeye Bar. - The materials of Buckeye Bar are evenly distributed along both banks of the Middle Fork. The pitrun material classifies as a GW-GM (well graded gravel with silt), and contains approximately 32 percent sand and 7 percent fines. The bar covers an area of approximately 1,104,000 square feet with an average depth to bedrock of 27.6 feet. It is estimated that the bar contains 1,128,000 cubic yards of material.

(7) Sardine Bar. - Sardine Bar generally lies within the channel of the Middle Fork and is rather small in size when compared to the other bars; also there are no clamshell investigations which are identified as being located specifically within Sardine Bar. For these reasons, the volume of aggregate contained in Sardine Bar will be included with the quantity of deposits which lie within the channel of the Middle Fork.

(8) Maine Bar. - Maine Bar is also rather small in size, and lies along the left bank of the Middle Fork just upstream of Sardine Bar. The material classifies as a GP-GM (poorly graded gravel with silt), and contains approximately 15 percent sand and 7 percent fines. The bar contains approximately 194,000 cubic yards of material, based on an area of 249,000 square feet and an average depth to bedrock of 21.0 feet.

(9) Philadelphia Bar. - Philadelphia Bar is located along the left bank of the Middle Fork, just upstream of Maine Bar. The material classifies as a GW-GM (well graded gravel with silt), and contains 28 percent sand and 10 percent fines. Given the average depth to bedrock of 23.7 feet and an area of 907,000 square feet, the bar contains about 705,000 cubic yards of material.

(10) Poverty Bar. - The materials of Poverty Bar are evenly distributed along both banks of the Middle Fork. The bar is located on a bend of the River just upstream of Philadelphia Bar. The material classifies as a SP-SM (poorly graded sand with silt), and contains 52 percent sand and 5 percent fines. Given an average depth to bedrock of 27.5 feet, and an area of approximately 1,152,000 square feet, the bar is estimated to contain 1,173,000 cubic yards of aggregate.

(11) Cherokee Bar. - Cherokee Bar is located upstream of Poverty Bar, primarily along the right bank of the Middle Fork. This is the longest of the bars identified as a potential source for concrete aggregate. The material classifies as a GC (clayey gravel with sand), and contains 22 percent sand and 12 percent fines. Based on an area of approximately 1,484,000 square feet with an average depth to bedrock of 18.0 feet, the bar is estimated to contain 989,000 cubic yards of material.

(12) River Channel. - The river channel of the Middle Fork American River is estimated to contain at least 968,000 cubic yards of material. This assumes an average thickness of six feet for the channel deposits and a channel area (exclusive of the bars) of 4,357,000 square feet. It is questionable if it is practical to excavate much of this material. Studies during PED will determine if this material can and should be used.

c. **Expected Extent of Aggregate Extraction.** - The total quantity of material available from the Middle Fork American River Bars and the river channel is about 9,573,000 cubic yards. If the river channel materials are ignored, the total quantity of material from the bars alone is about 8,605,000 cubic yards, or about 27 percent greater than the quantity required for the dam. Hence, if the Middle Fork American River bars are available for use and are determined to be the best source of aggregate, all of the material in the bars would be required. The channel material would probably be left in place.

d. **Suitability of Bar Deposits for Concrete Aggregate.** - As can be seen from the plots of the pit run gradations shown in Figures 5 through 14, as well as the band of gradation recommended by the American Concrete institute (ACI) shown on the plots, the bar deposits along the Middle Fork are generally well suited for use as aggregate for RCC. However, some adjustments to the pitrun material will be necessary, since an average of approximately 23 percent of the aggregate will need to be crushed to bring the gradation in line with the upper portion of the aggregate band recommended by ACI for RCC aggregate. As can be seen from the gradation plots, Kennebeck and Poverty bars contain more sand sized materials than what is recommended by the ACI aggregate band. There does not appear to be any systematic variation in the distribution of particle sizes with depth within the bars or between bars along the Middle Fork, due probably to the origin of the deposits from upstream hydraulic mining, as well as later disturbance of the bars by dredging. Analysis of the Figure 15 Bar Gradation Summary Data supports several conclusions regarding the Middle Fork bar deposits. 36 percent sand is a reasonable percentage for RCC aggregate, especially since crushing of the oversize coarse aggregate will also produce additional sand. No supplemental sand source appears to be necessary for the Middle Fork American River aggregates. The existence of 8 percent fines in the bars will probably require wasting of about 3 to 4 percent fines. The volume statistics indicate that the volume of aggregate in each bar is relatively evenly divided, rather than being concentrated in a few bars.

e. **Aggregate Excavation Methods.** - Due to the annual flooding of the aggregate bars in the river, it is assumed that the bars will be accessible between 8 and 10 months each year. This will require an aggregate production during this period of about 30,000 tons per day to provide an adequate supply for year-round RCC production. The average depth of material in the bars ranges from 18 to 31 feet, with an overall average of 26 feet. The type of equipment and procedures that a contractor would use on the bars is difficult to predict exactly, but some reasonable judgements can be made. Since most of the aggregate is underwater, draglines are the preferred type of equipment for excavation. Draglines have relatively high capacities and dig effectively underwater. Based upon the required production rate, three to four large draglines, working 12 hours per day, would be needed. To avoid congestion at each bar, a maximum of two draglines could work each bar. Two bars would be worked simultaneously. The draglines would dump their buckets into a portable track-mounted primary processing unit. This unit would consist of a hopper, primary screen, jaw crusher, and conveyor. Aggregate would be screened to remove oversize material (large boulders), crushed to 3-inch maximum size, and conveyed by a series of portable conveyors to the primary conveyor.

f. **Disposal of Waste Material.** - The disposal of waste products from the processing of the pitrun sand and gravel may be one of the problems associated with use of bar deposits along the Middle Fork. The fine grained by-products of aggregate washing cannot be dumped back into the channel after processing due to restrictions on turbid discharges into streams. Waste stockpiles for such fine grained material must also be protected from erosion. If a quarry is employed to supplement the bar aggregates, the waste fines could be used for rehabilitation of the quarry site after construction. However, it is likely that a large percentage of the fines will be used in the RCC. Off-site disposal will be needed for the remainder of the fines. Most of the oversize rock probably can be left in the bar deposit.

g. **Control of Water Turbidity.** - The environmental restrictions associated with control of effluent quality during aggregate excavation and processing will require specialized state-of-the-art equipment and treatment methods. Numerous settlement ponds will likely be needed in the Middle Fork American River to trap sediment and allow removal from the flow using flocculating agents. For the aggregate plant(s), a closed-end system will be needed to process and recirculate the wash water.

h. **Access to Bars.** - Space along the bottom of the channel is restricted along the Middle Fork, as can be seen in Photos 4,5 and 6. Roads will need to be developed from existing county and state roads to the bars to haul equipment, along the conveyors for conveyor maintenance, and to the aggregate and concrete plants. Existing road access to the bars will likely be used as much as possible, improving and widening the roads, as shown on Figure 1. Areas will need to be developed for material stockpiles, vehicle maintenance facilities, and effluent control equipment. The best location for some of these areas will likely be near Mammoth Bar.

4. <u>Middle Fork American River Bar Processing.</u> - Information on processing and transportation for the Middle Fork American River aggregates, aggregate storage areas, and supplementary sand sources, follow. For this study, it has been assumed that RCC will be placed 260 days each year, with a slow start due to constricted placement area in the foundation, and slow placement at topping out, due to the narrow crest width. This mandates a minimum concrete placement capacity of 12,000 cubic yards per day, assuming two 10-hour shifts, six days a week. Cost of the aggregate delivered to the damsite by conveyor has been conservatively estimated to not exceed $ 7.40 per ton.

a. **Location and Extent of Transport and Processing Areas.** - Figure 1 shows the damsite, the Middle Fork American River bars, a possible route for a conveyor and processing/storage locations. Photo 1 shows the damsite area and probable location of an RCC plant. There are a number of alternatives that a contractor could select for transport routes and processing/storage areas. That shown in the Figure is only one alternative, but the general concept of these alternatives is the same.

b. **Aggregate Storage.** - Aggregate storage for the Middle Fork American River Bars is clearly the most challenging problem for RCC production, due to the lack of relatively flat terrain in or near the American River canyon. Since the river bars will be flooded during a portion of each year, RCC production would have to stop unless sufficient storage of aggregates is provided for several months of RCC production. Three solutions are apparent, as follows:

(1) Provide a minimum of two months of aggregate storage somewhere near the damsite. This would require the construction of extensive cuts and fills along the steep canyon walls to form working, storage and plant areas. About 40 acres of storage would be required to be distributed along the conveyor transport route. The location of these areas is the most difficult technical problem for use of the bars for concrete aggregate. There is presently no obvious location for these areas.

(2) Provide only minimal aggregate storage, and place RCC only during aggregate production months (probably 8 to 10 months), using the same plant

capacity as anticipated for year-round RCC production. This avoids the aggregate storage difficulties, but would extend RCC construction into a third year.

(3) Provide only minimal aggregate storage, and place RCC only during aggregate production months (probably 8 to 10 months), using higher plant capacity than anticipated for year-round RCC production. This avoids the aggregate storage difficulties, would allow RCC construction to be completed in two years, but would necessitate higher capacity aggregate excavation, processing, and transport equipment, as well as a higher capacity RCC plant.

Most of the aggregate processing could be performed in a plant located above Mammoth Bar. This would allow storage of finished aggregate in areas distributed along the primary conveyor to the RCC plant at the damsite. These storage areas could consist of a stacking conveyor to divert aggregate from the primary conveyor to a stockpile. When aggregate was ultimately needed from this stockpile, a reclaim tunnel conveyor would transport aggregate from beneath the stockpile back to the primary conveyor.

c. **Processing Facilities and Operations.** - The aggregate processing plant could be located near the damsite or nearer the site of excavation. Figure 1 shows two likely locations for an aggregate plant. In either location the plant will have to be located on benches cut into the canyon walls. Aggregate plants require a considerable amount of acreage, but a much smaller space than that required for aggregate stockpiles. The processing consists of primary screening, primary crushing, screening into coarse and fine fractions, secondary crushing of coarse aggregate, secondary screening of coarse and fine aggregate, washing, possibly sand manufacturing by crushing, and stockpiling. Primary screening and crushing would probably be done at or near the site of excavation. The capacity of the aggregate plant ordinarily is matched with the RCC mixing plant capacity. If advance stockpiling of large quantities of aggregate is required for year-round RCC placement, the aggregate plant capacity will exceed the RCC plant capacity by up to one-third.

d. **Sand Source.** - The usable portion of sand is about 36 percent of the Middle Fork bar pit run material, which is adequate for RCC production. Additional sand could be made as a by-product of the coarse aggregate crushing process, or by utilizing a separate crushing process to produce sand-sized aggregate from the Middle Fork materials.

Sand may be available from the Chevreaux Quarry about 12 miles north of the town of Auburn. Tests were conducted by the USBR on crushed rock samples (metavolcanic breccia) of this material. Chevreaux produced between 5,000 and 13,000 cubic yards of concrete aggregate sand per year in 1971, 1972, and 1973 for work done by the USBR at the Auburn Dam site.

Sand (Amphibolite) may also available from the Cool Quarry about 1-1/4 miles north of the town of Cool. This source may be Government owned.

Opening a rock quarry to produce all of the necessary aggregate for an RCC dam is a possibility. Under this scenario the required amount of sand would be produced at the quarry site. Sands can be produced by several different methods of processing (cone crusher, hammer mill, rod mill) and all are physically suitable, but the cone crusher produces the most spherical particles, and is the method usually selected by contractors.

5. <u>Alternate Aggregate Sources.</u> - Alternatives to the Middle Fork American River Bars for concrete aggregates are described in the following, including damsite materials, North Fork American River aggregates, potential onsite quarries, access, extraction, transportation and remediation for a quarry, alternate distant aggregate sources, and aggregate source considerations.

a. **Damsite Materials.** - A variety of material will be available for possible use as concrete aggregate from the damsite. This includes weathered rock from the foundation excavation, streambed excavation consisting of a mixture of cofferdam material and alluvial aggregate, and cofferdam material. The quantity of these materials varies, but is substantial. The quality of these materials, from visual inspection only, is suspect. For the purposes of this study, these materials have been assumed to be of inadequate quality for use in RCC. During PED, use of any of these materials will be strongly pursued, in order to reduce the quantity of either quarry or bar excavation further upstream. Photo 1 shows this area.

b. **North Fork American River Aggregates.** - There are several sand and gravel bars along the North Fork of the American River that lie between the backwaters of Lake Clementine and the Ponderosa Bridge, a distance of about 4 miles, shown on Figure 2. The river mile distance of this source from the damsite is about 10 miles, as compared to 5 river miles between the damsite and Mammoth Bar. Limited field explorations by USBR were performed with a backhoe, and stadia survey and cross sections were completed to Ponderosa Bridge to calculate surface areas of the bars. It was estimated that there is between 2 and 4 million cubic yards of aggregate in these bars. No estimate was made as to the percentages of sand and gravel in this total. Although not containing enough material to meet the total requirements for the flood control dam, these bars could supplement the Middle Fork American River or other source. An extensive exploration program would have to be carried out to determine the quality and quantity of materials in this source. As with the Middle Fork American River bars, an extensive series of settling ponds and other measures would have to be utilized to minimize turbidity caused by the release of fines during excavation. However, little room is available upstream of Lake Clementine for such ponds. At present there is no access except for Ponderosa Way at the bridge, up over the Forest Hill Divide (some 1400 feet rise in elevation) to Forest Hill Road, and thence about 16 miles to the site. Due to the lengthier distance to the damsite compared to the Middle Fork American River bars, and problems to be resolved similar in nature to the Middle Fork bars, it is unlikely that this source of aggregate will be used. This source will be further investigated during PED to determine final potential for use.

Little information is presently available regarding aggregate that may be lying beneath the waters of Lake Clementine, created by the construction of North Fork Dam from 1937 to 1939. The primary purpose of the dam was to store the debris produced by hydraulic mining in the upstream reaches of the river. However, very little upstream hydraulic mining activity has occurred since completion of the project, resulting in only a fractional amount of expected material to be deposited in the reservoir. As seen on topographic maps, the slope of the reservoir walls ranges between 30 and 40 degrees, to produce a steep, narrow, v-shaped canyon with little storage space at or below river level. Only a few small bars are shown along the river between the dam and the confluence with the Middle Fork, and the assumption can be made that the stretch of river from the dam up to the backwaters of Lake Clementine could

contain similar deposits. Pre-1939 topography shows the river to range in width from 30 to 80 feet in the vicinity of the dam. During the pre-construction explorations, three holes were drilled in the river just upstream of the dam axis. These holes encountered gravels ranging in thickness from 12 to 19 feet overlying bedrock. The cross-sectional area of the gravel is estimated to be about 500 square feet. The river distance from the dam to the backwater of Lake Clementine is about 21,000 feet. These numbers could indicate that a maximum of 389,000 cubic yards of aggregate exists in Lake Clementine. An unknown amount of material has undoubtedly been deposited in the reservoir by limited hydraulic mining and by seasonal flooding. Even if this doubled the amount of aggregate estimated above, the total quantity of material in the lake appears to be insufficient for serious consideration as aggregate for the dam. Extracting this relatively small amount of aggregate from significant depth underwater would be extremely expensive, would create an enormous turbidity problem, and would be technically difficult to accomplish. This source of aggregate will be studied further during PED, but it is considered an unlikely source of aggregate the flood control dam.

c. **Potential Onsite Quarry Sites.** - There are several potential quarry sites in the vicinity of the proposed damsite, as discussed below. Figures 1 and 3 show these locations.

(1) Oregon Bar Pluton. - A brief review of the available information was made to determine if the quartz diorite of the Oregon Bar pluton would be a suitable aggregate for RCC. Due to extensive shearing and deep disintegration of the rock, the Oregon Bar pluton is not considered a potential source of concrete aggregate.

(2) RM 22.4 Quarry Site. - A potential quarry site is located in the N 1/2 of the NE 1/4, Sec 14, T. 12 N., R. 8 E., (USGS Auburn, 7.5 minute quadrangle map), in the downstream portion of the left abutment for the earlier proposed dam at RM 22.4. A feasibility study was conducted at this site for a concrete dam in 1942. Little exploration was done on the left abutment, but the original geologic report refers to it as a fairly hard rib of amphibolite schist. The mapping done by the USBR refers to it as amphibolite. The rock is hard and dense with prominent but discontinuous joint sets. The numerous joints would probably be an advantage in a blasting program in that very few oversize pieces would be produced. Overburden on the left abutment is relatively light and numerous outcrops of rock may be seen. While the rock appears suitable for aggregate and is in close proximity to the dam site, the environmental problems at this site may be too large to overcome. The quarry site will be in full view of the homes built around Robie Point at the edge of the town of Auburn, a distance of about 2500 feet directly across the river from the site. Working the quarry for a period of several years in full view of the town, with the expected noise problem, may not be acceptable. Mitigation of the huge scar that would be left in the hillside would be difficult. This source may be considered further during PED, but it should be considered an unlikely source of concrete aggregate at this time.

(3) Amphibolite at the Cool Quarry. - This potential quarry site is pictured in Photos 2 and 3, and Figures 1 and 3. In 1981 the USBR conducted an exploration program of core drilling to find an acceptable quarry site for the Auburn Project. This area is located in the SE 1/4 of the SW 1/4, Sec 6, and also the SW 1/4 of the SE 1/4, Sec 6, T. 12 N., R. 9 E. (USGS Auburn, 7.5

minute quadrangle map). It is located immediately to the west of the existing Cool Quarry, which is leased by Spreckels Limestone and Aggregate. Seven holes were drilled during this investigation, six vertical and one near horizontal, for a total of 1254.9 feet. The rock encountered in these drill holes has been described on the USBR drill logs as fine-grained metavolcanics, metatuff-breccia, and metatuff with some minor diorite and latite dikes. Amphibolite is the collective term generally used when referring to these rocks. Descriptions of the core show some pieces to be up to 4.7 feet long, but most are under 2.0 feet long. This degree of fracturing may be an advantage to a quarry contractor in that lighter power loads and wider spacing on the blast holes could be used, thus decreasing the cost of excavation and crushing. The drill holes are all located along the eastern edge of the nose of the ridge. By extending the proposed quarry to the west into the SW 1/4 of the SW 1/4 of Sec 6, there should be enough material to supply sand and coarse aggregate for the RCC dam. Twenty-six rock core samples from the quarry were sent to the USBR Denver Lab for thin section analysis. The report states that the samples appear to be petrographically suitable for use as concrete aggregate. However, the report also states that the samples have to be submitted to the Lab for physical properties testing as a proposed concrete aggregate source. Physical testing of this material for use in concrete was never done, nor was an official quarry report done. While environmental concerns will still exist at this site, they may not be as significant as the other quarry sites. The quarry is adjacent to a large existing quarry, and would be about 2 miles east of town and should be out of sight. This site is considered the most feasible for a quarry operation for concrete aggregate.

(4) Quarry Access, Extraction and Transport. - Use of the Cool Quarry Amphibolite is assumed for discussion of access, extraction and transport to the damsite. Since the quarry is out of the American River streambed, there is no interruption in production during each year due to high water. Quarry operations can be carried on year-round, and consequently, extensive stockpiles are not required for adequate RCC production. Extraction will be by standard drilling and blasting methods. In general, the haul roads at any quarry are confined to the boundaries of the quarry. In this way the roads are taken out as the quarry is lowered to its final elevation and there are no road scars left on the surrounding hillsides. The rock would probably be processed at or near the quarry, and conveyed to the primary conveyor to be transported to the concrete plant or another aggregate plant. Potential quarry access is shown on Figure 1. Cost of the Cool Quarry Amphibolite delivered by conveyor to the damsite has been conservatively estimated to not exceed $10.00 per ton.

(5) Quarry Remediation. - Remediation of quarry sites is difficult at best. It is unlikely that the gaping hole typically left by quarry operations could be easily and cheaply removed or hidden. Any spoils remaining could be pushed into the quarry floor or benches and spread to create a small hummocky topography. Soil or fines from bar processing could be spread over the top of this material and seeded to produce vegetation.

d. **Alternate Distant Aggregate Sources. -**

(1) Bear River and Chevreaux Quarry. - The deposits along the Bear River, located on highway 49, north of the Auburn damsite, could provide a large quantity of good quality aggregate for use in RCC. The largest deposit is privately owned and operated by Chevreaux Concrete. Their coarse aggregate is produced from blasting and crushing quarry rock located near the river(see Chevreaux Quarry below). Sand is obtained from dredging in Lake Combie on the Bear River. Available materials are estimated to be well in excess of what would be needed for a concrete dam at Auburn. Aggregate could be trucked to a concrete plant at the Auburn damsite, a distance of approximately 11 miles, at a cost of $9 per ton delivered. Rail transport from this source is not feasible.

(2) Mississippi Bar on the American River. - Mississippi Bar is a large bar deposit below Folsom Dam, consisting of sands and gravels which were dredged for their gold content from 1917 to 1949. The bar is located on the south shore of Lake Natoma, 4.6 miles SW of Folsom Dam. The Mississippi Bar deposits are owned by the Government and were used to supply concrete aggregate for the construction of Folsom Dam. These deposits were being considered for use as an alternate source of concrete aggregate for Auburn Dam by the USBR in 1967. The Government has leased the bar since 1957 to commercial suppliers of concrete aggregate. Currently the deposits are being mined by Teichert Aggregates. The pit run gradation of the Mississippi Bar deposits would be generally well-suited for use as RCC aggregate. The amount of material currently available at the site is in excess of 10,000,000 cubic yards. These deposits could be hauled to the concrete plant near the Auburn damsite, a distance of approximately 18 miles, at a cost of about $8 to $9 per ton. Rail transportation is a possibility at this source.

(3) Yuba River. - The deposits along the Yuba River near Marysville consist of vast expanses of dredger piles. Such deposits are similar to those along the American River and at Mississippi Bar, but much larger in total volume. Most of these deposits along the Yuba River are government owned, although this ownership is sometimes disputed, and are currently being processed by several commercial aggregate companies. Baldwin Construction has the largest on-going processing operation along the Yuba River. Available material at their site is in excess of 10,000,000 cubic yards. The material could be trucked to a concrete plant near the Auburn damsite, a distance of approximately 40 miles, at a delivered cost of about $10 per ton. The quality of this aggregate is well established, having been investigated for the Marysville Dam Project in the 1970's and the Cache Creek Project. Rail transportation of aggregate may be a possibility at this source.

(4) Commercial Quarries. -

(a) The Chevreaux Quarry is located about 2 miles north of the town of Meadow Vista, approximately 12 miles north of Auburn in Placer County. It is in T41N, R9E, SW 1/4 Sec 30, MDBM (Lake Combie 7.5 Quadrangle). This material has been used for road base, drain rock, ballast, filter media, road rock, fill material, concrete aggregate, and landscape material (the larger as riprap). The operator estimates reserves of 80 million tons. SPD Laboratory classified this rock as a metavolcanic breccia. The cost of this aggregate is described above under Bear River aggregates.

(b) The Cool Quarry is located approximately 1-1/4 miles north of the town of Cool on the east side of Highway 49 in El Dorado County. It is in T12N, R9E, NE 1/4 Sec 7, MDBM (Auburn 7.5 Quadrangle). This is not really a distant source, but is described in this portion of the report because it is considered a commercial source. The material produced has been used for refining sugar, glass manufacture, cosmetics, roofing material, road base, concrete aggregate, riprap, and various chemical applications. The operator estimates reserves of 12 million tons of marble and 100 million tons of metavolcanic breccia. SPD Laboratory classified the rocks as a marble and a metavolcanic.

e. **Aggregate Source Considerations.** - Commercial aggregate sources are used for most small to medium sized concrete projects, where thermal and structural properties of the concrete are not driving elements of the design. Large mass concrete projects, however, typically involve several years of effort to determine the complex thermal and strength properties of the concrete. Because of this, the aggregate source for these kinds of projects are invariably determined and provided by the Government. The alternative, allowing the Contractor to select a source, could result in use of a concrete with different properties than those used for design. This does not necessarily rule out all distant sources of aggregate, since some of these sources are apparently already Government owned.

Transportation by truck on existing public roads and highways is available to all of the distant commercial aggregate sources discussed in this report, but additional roads will need to be constructed to any aggregate plant near the damsite. The logistics of this truck transport are truly formidable. To facilitate the placement of 5,000,000 within two years, approximately 700 to 900 truck trips would be needed to deliver aggregate each day, or about 40 truckloads per hour. The commercial suppliers contacted for this report felt that obtaining the required haul permits from Caltrans would not be a problem. However, the ability of the existing state and county roads to withstand this punishment is questionable. Rail transportion should be strongly considered if distant aggregate source(s) are needed to construct the flood control dam.

6. <u>Conclusions and Recommendations.</u> -

a. Refinement of most of the ideas expressed in this report will be done during PED. This applies particularly to sources of aggregate and transportation of aggregate.

b. Whatever aggregate is selected during PED for use, it must be a Government-supplied source.

c. Adequate quantities of high quality aggregates are present in the Middle Fork American River bars. If this source of aggregate is used, all of the bars from Mammoth to Cherokee Bar will be required. No supplemental sand source will be required.

d. Alternate aggregate sources will be considered early in PED. The aggregate sources that are the most likely to be investigated in detail during PED are:

Middle Fork American River Bars
Cool Quarry Amphibolite
Mississippi Bar - American River
Yuba River
Old Cool Quarry

Each of these sources has advantages and disadvantages that will be analyzed in detail in PED.

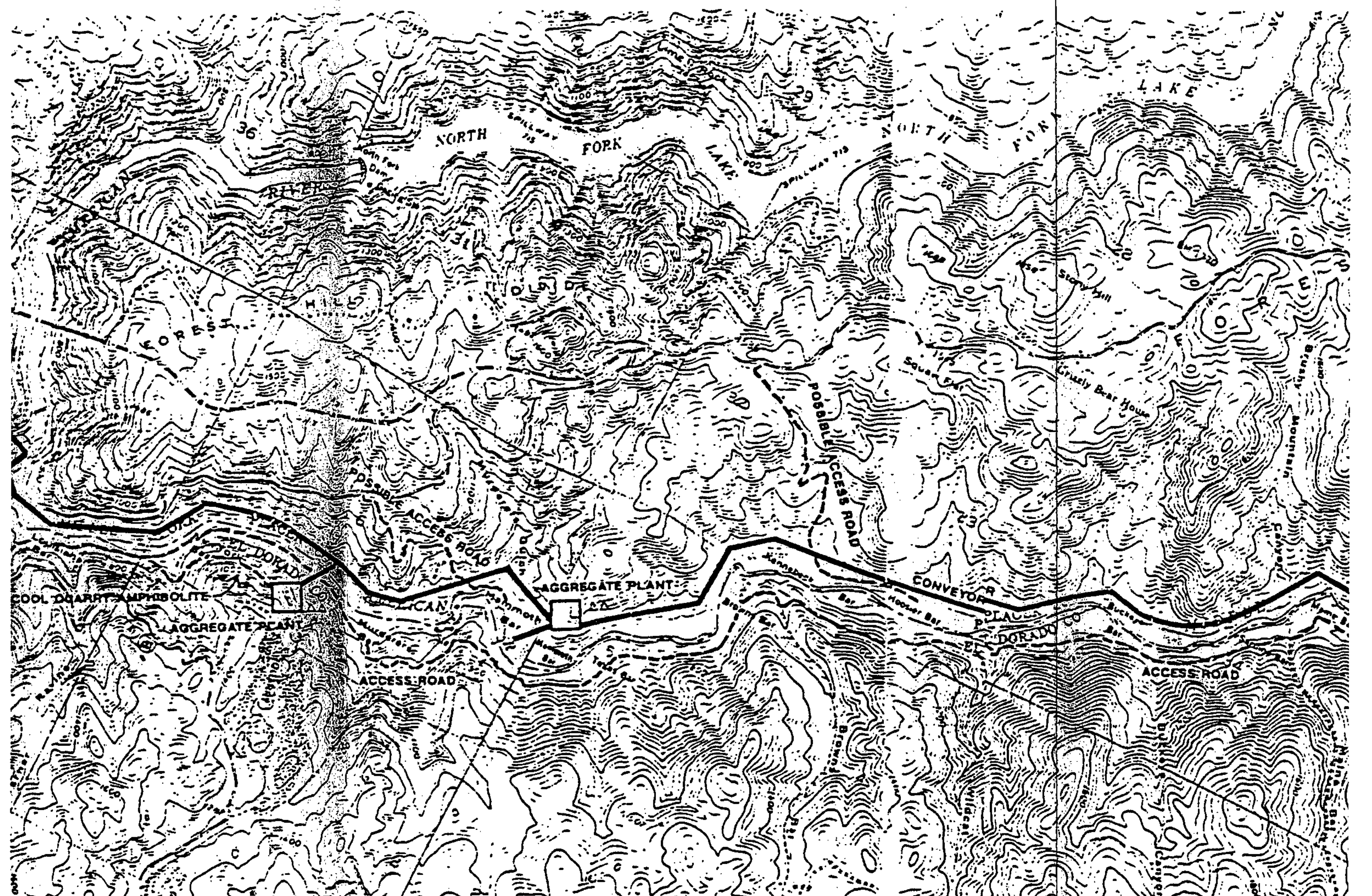
e. Conveyor is the most likely method of aggregate transport, except that rail is a possibility for the Mississippi Bar materials. Truck transport is unlikely because of cost, but the logistics involved with hundreds of trucks per hour delivering aggregate to a rural community are probably more of a problem.

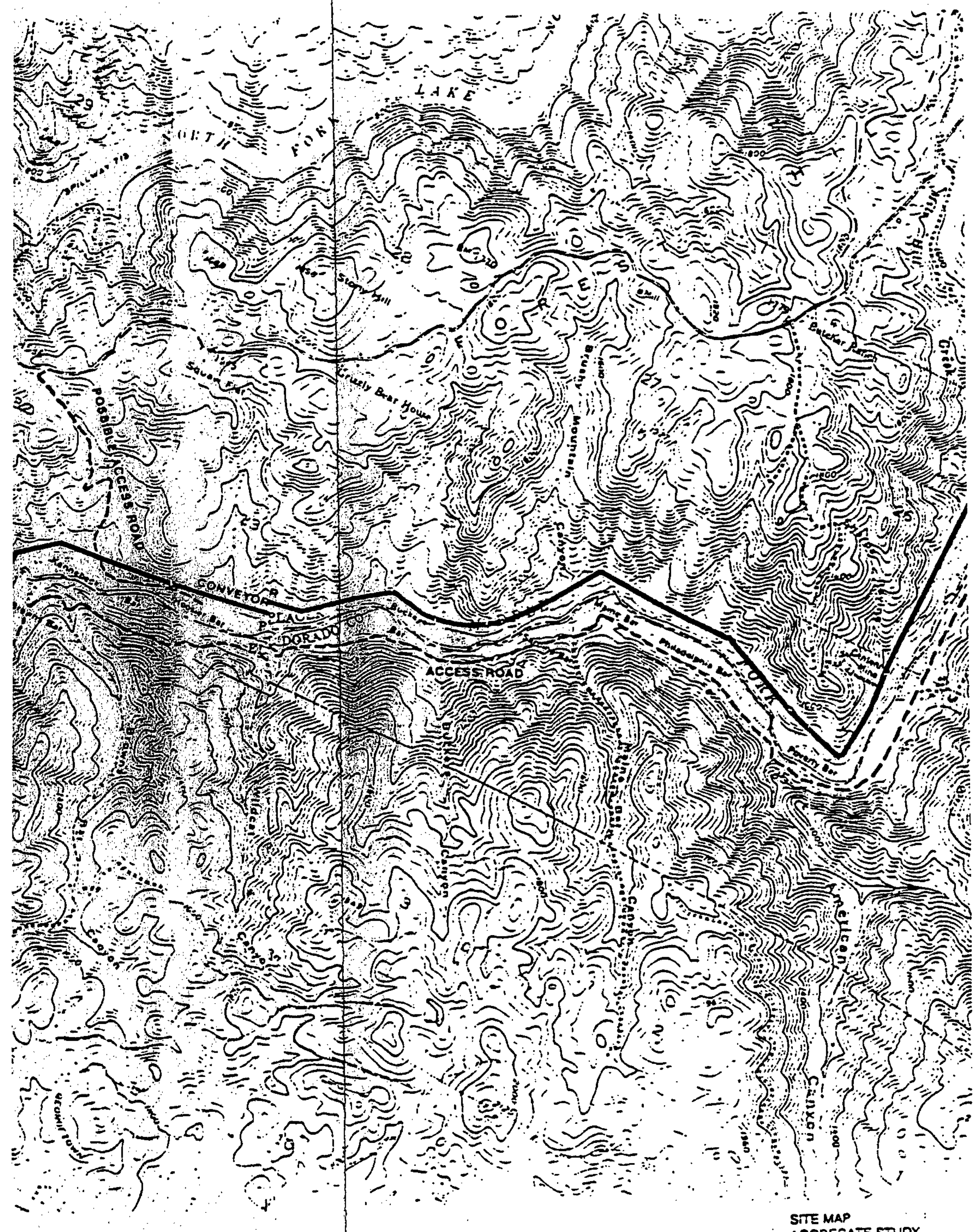
f. Close coordination between Engineering and Planning Divisions will be needed during PED to identify and deal with environmental consequences of and remediation for use of aggregate sources, particularly:

(1) Turbidity control;
(2) Waste material disposal;
(3) Quarry remediation;
(4) Transport and processing site remediation;
(5) Stream hydraulics and bar removal.

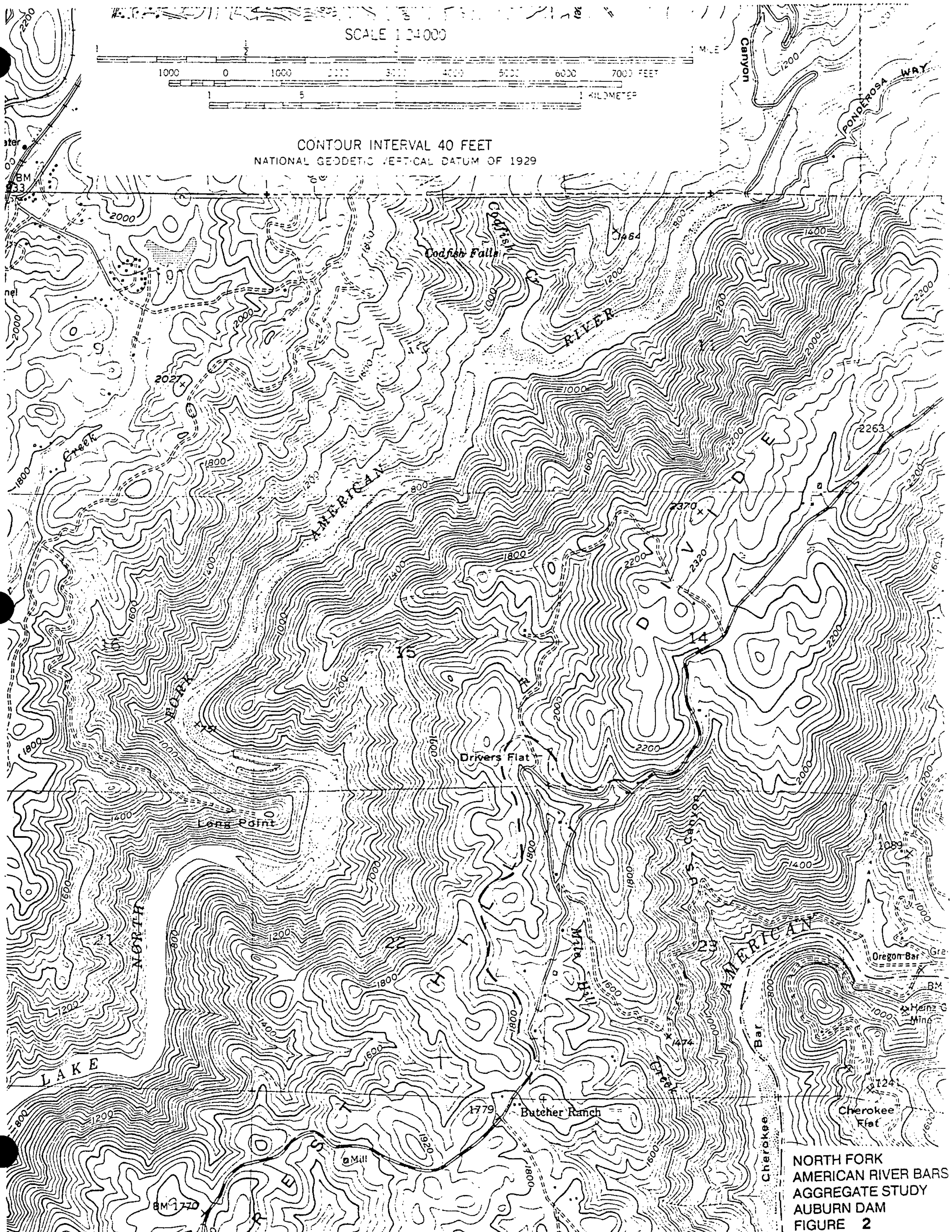
g. Studies of concrete aggregate must be initiated aggressively at the very beginning of PED, to verify as rapidly as possible the source of aggregate for the flood control dam.

AUBURN
CONVEYOR
COOL QUARRY AMPHIBOLITE
AGGREGATE PLANT
RM 22.4 QUARRY SITE
CONCRETE PLANT
AUBURN DAMSITE
OREGON BAR PLUTON QUARRY SITE
FOREST HILL
RIVER
Cool
Creek
Reservoir

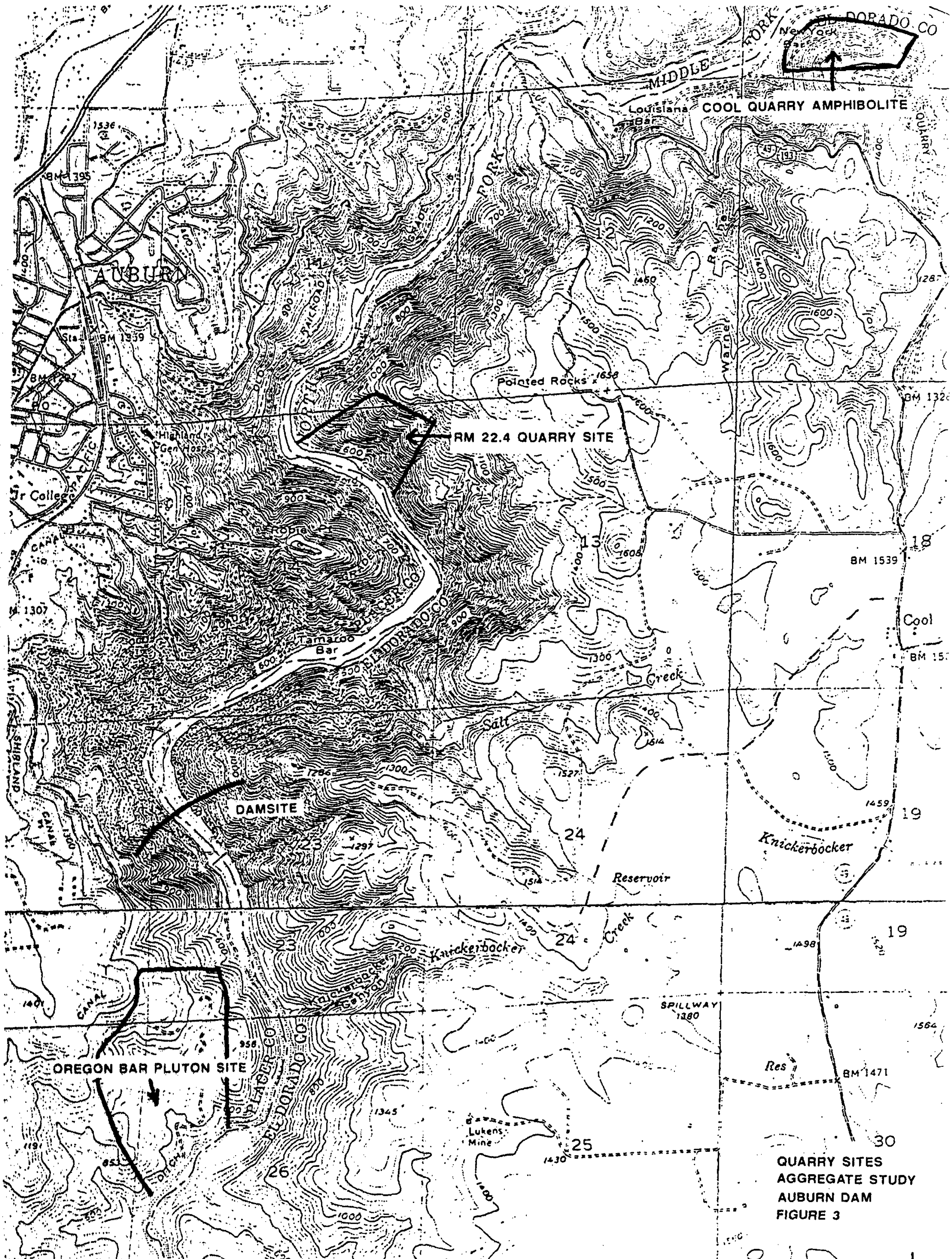

NORTH
FORK
LAKE
RIVER
HILL
FOREST
POSSIBLE ACCESS ROAD
AGGREGATE PLANT
CONVEYOR
COOL QUARRY AMPHIBOLITE
AGGREGATE PLANT
ACCESS ROAD
PLACER
EL DORADO CO
ACCESS ROAD
Cool



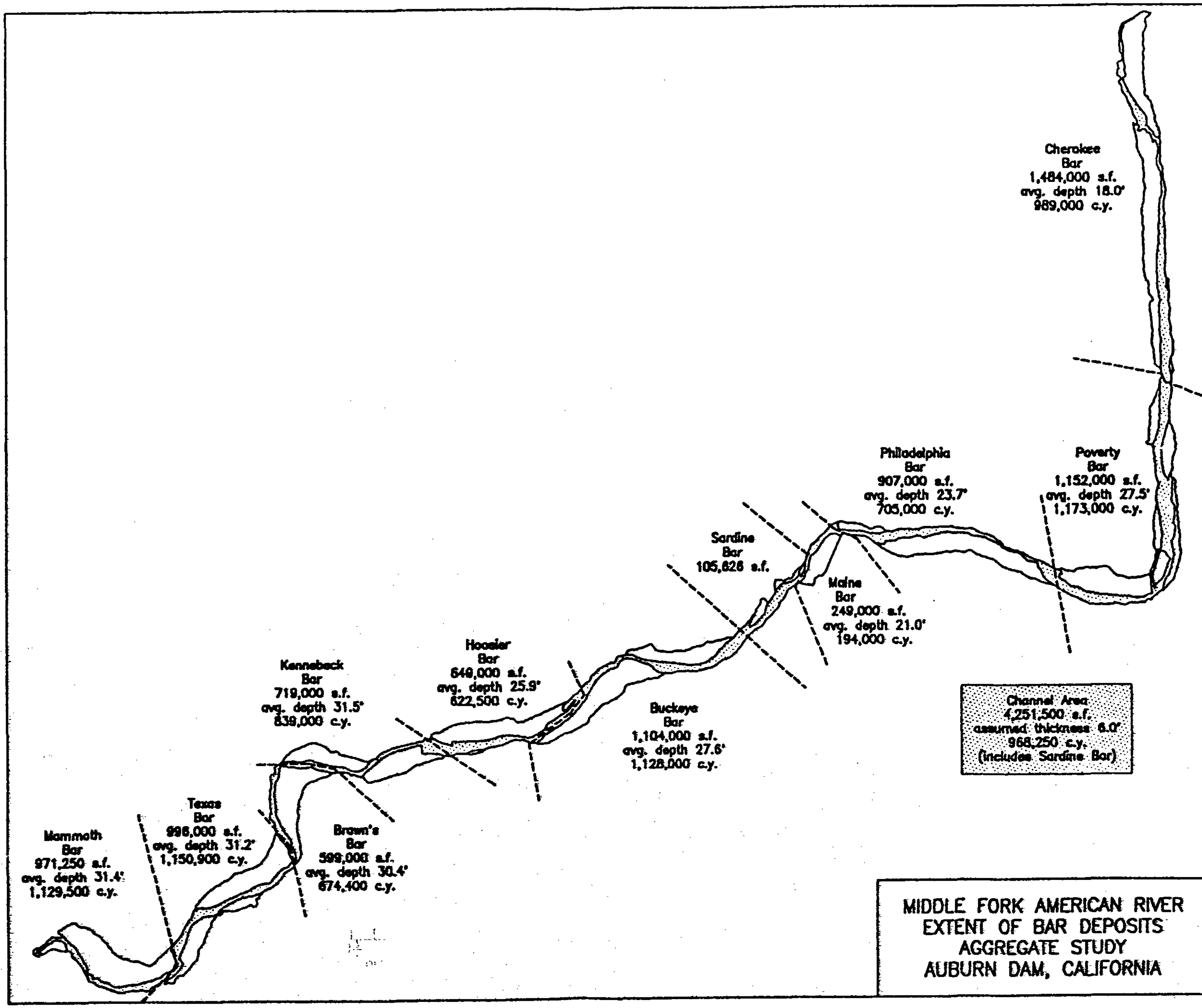
SITE MAP
AGGREGATE STUDY
AUBURN DAM
FIGURE 1



NORTH FORK
AMERICAN RIVER BARS
AGGREGATE STUDY
AUBURN DAM
FIGURE 2

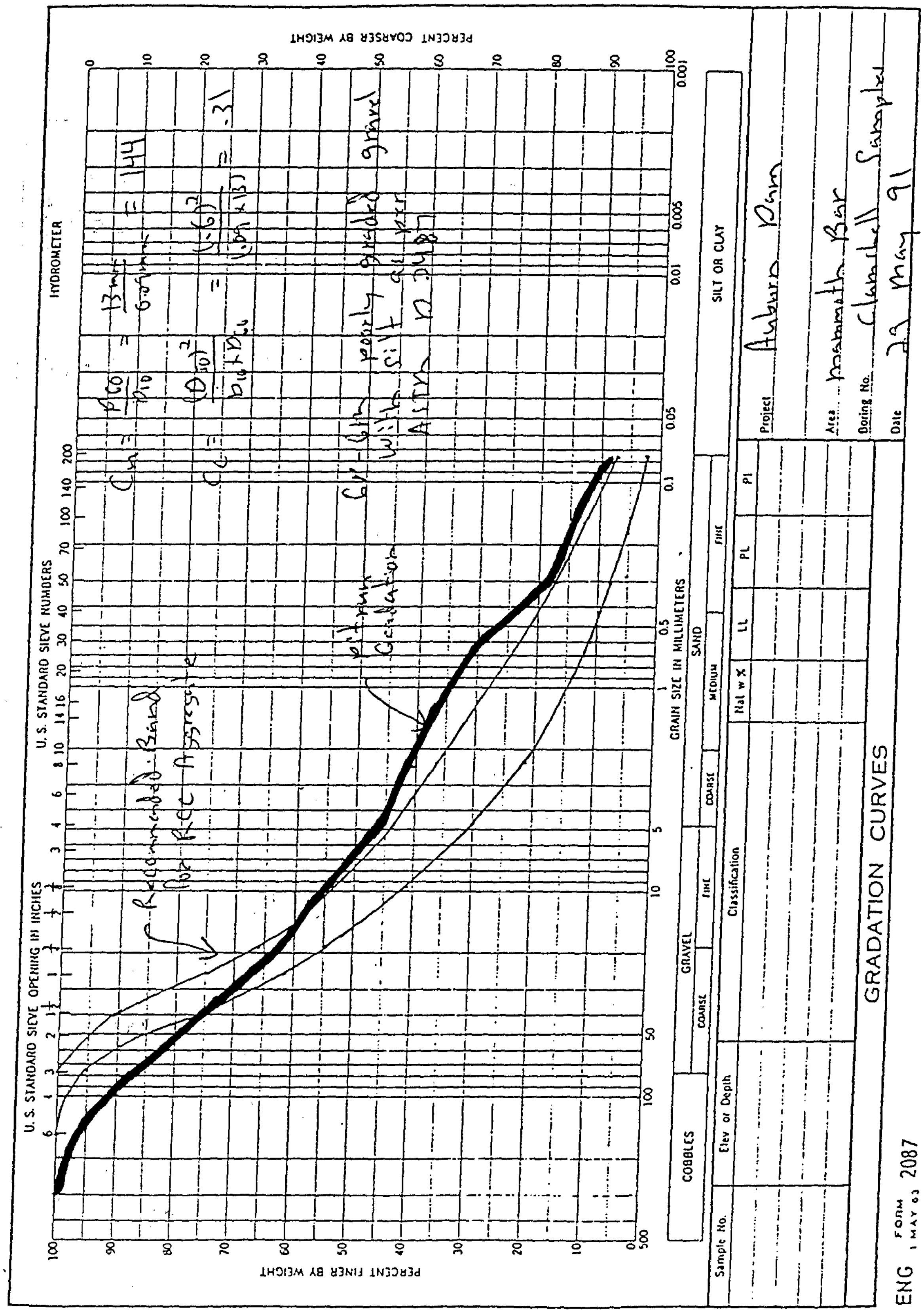


QUARRY SITES
AGGREGATE STUDY
AUBURN DAM
FIGURE 3

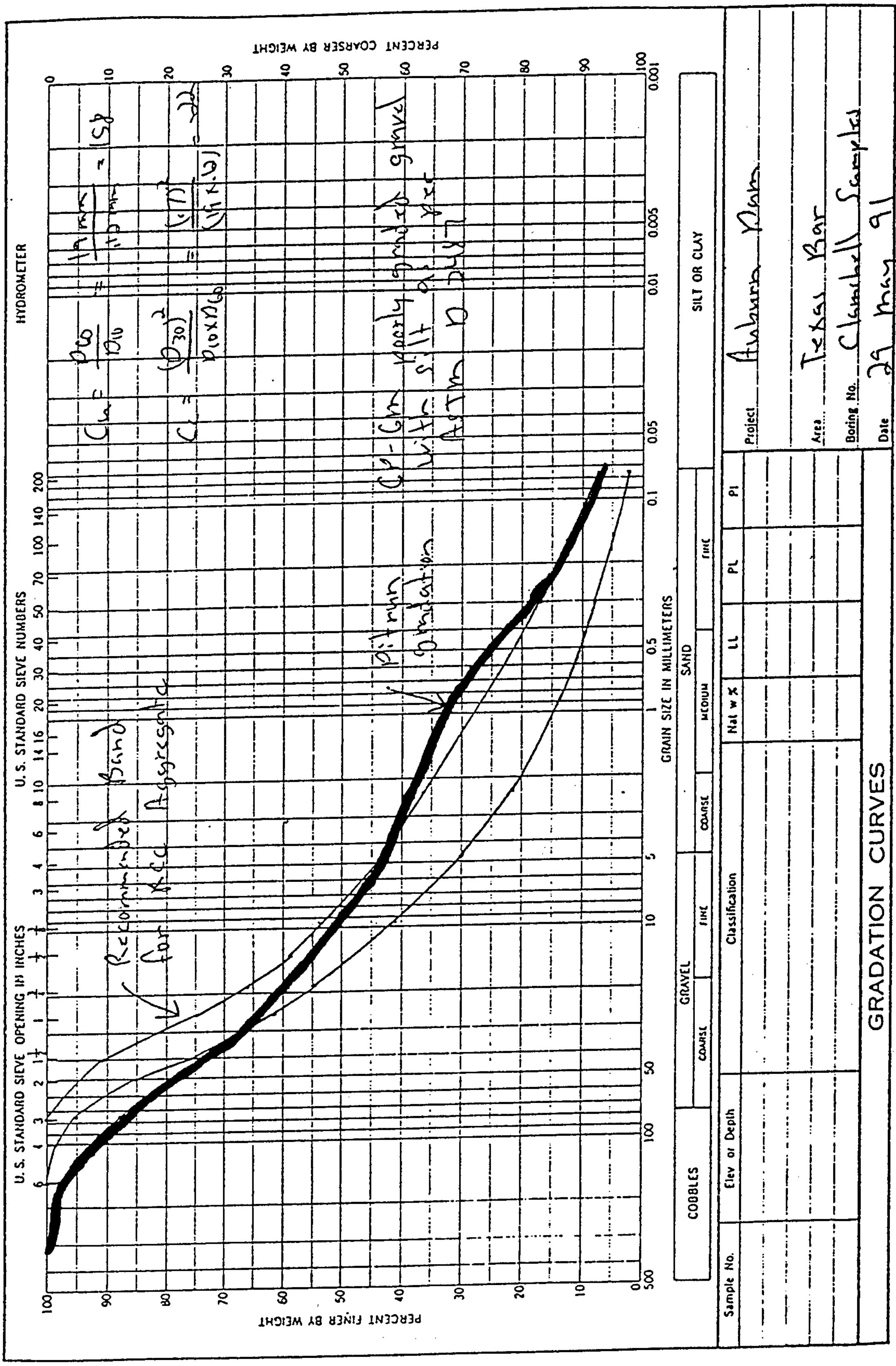


MIDDLE FORK AMERICAN RIVER
EXTENT OF BAR DEPOSITS
AGGREGATE STUDY
AUBURN DAM, CALIFORNIA

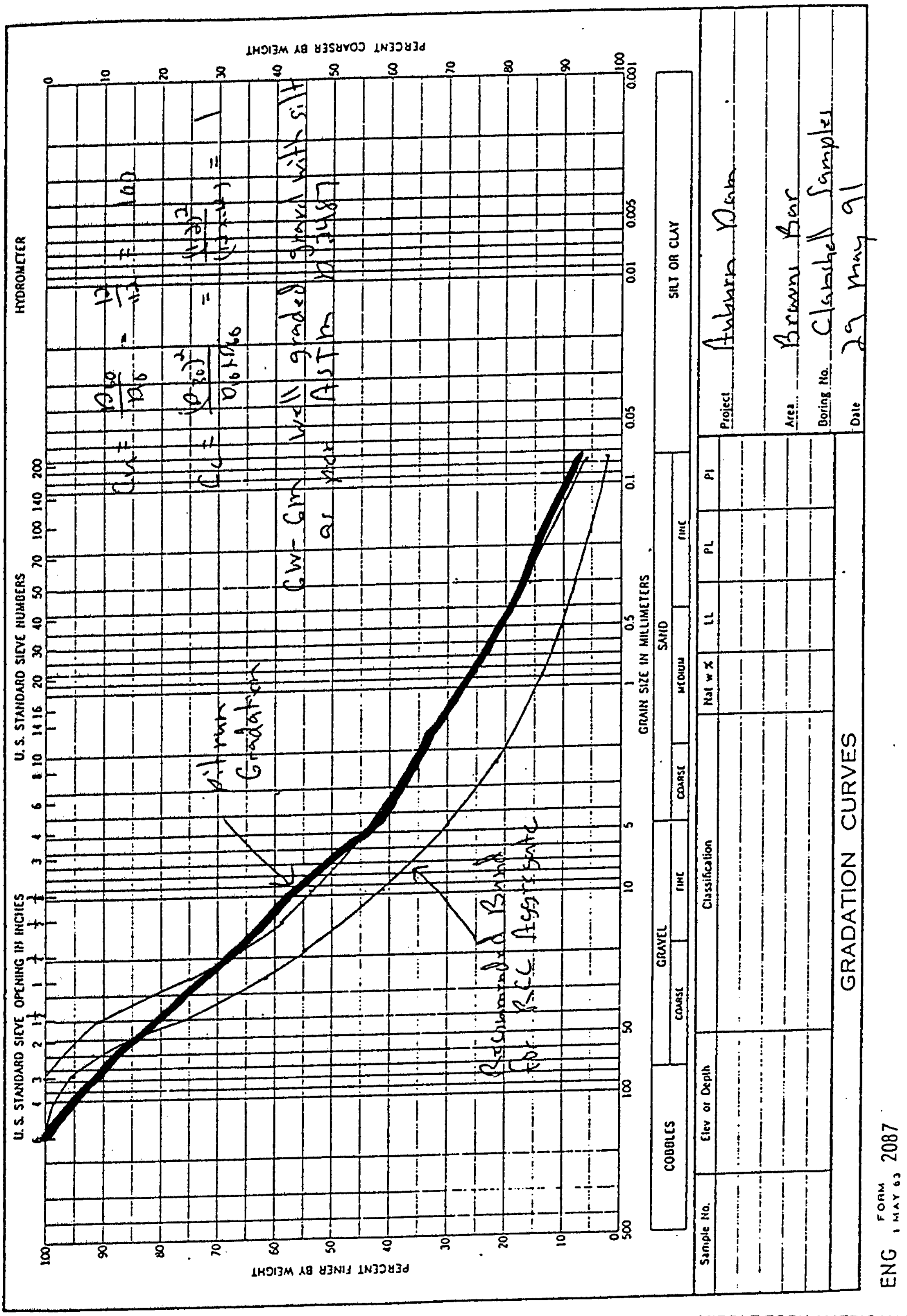
FIGURE 4



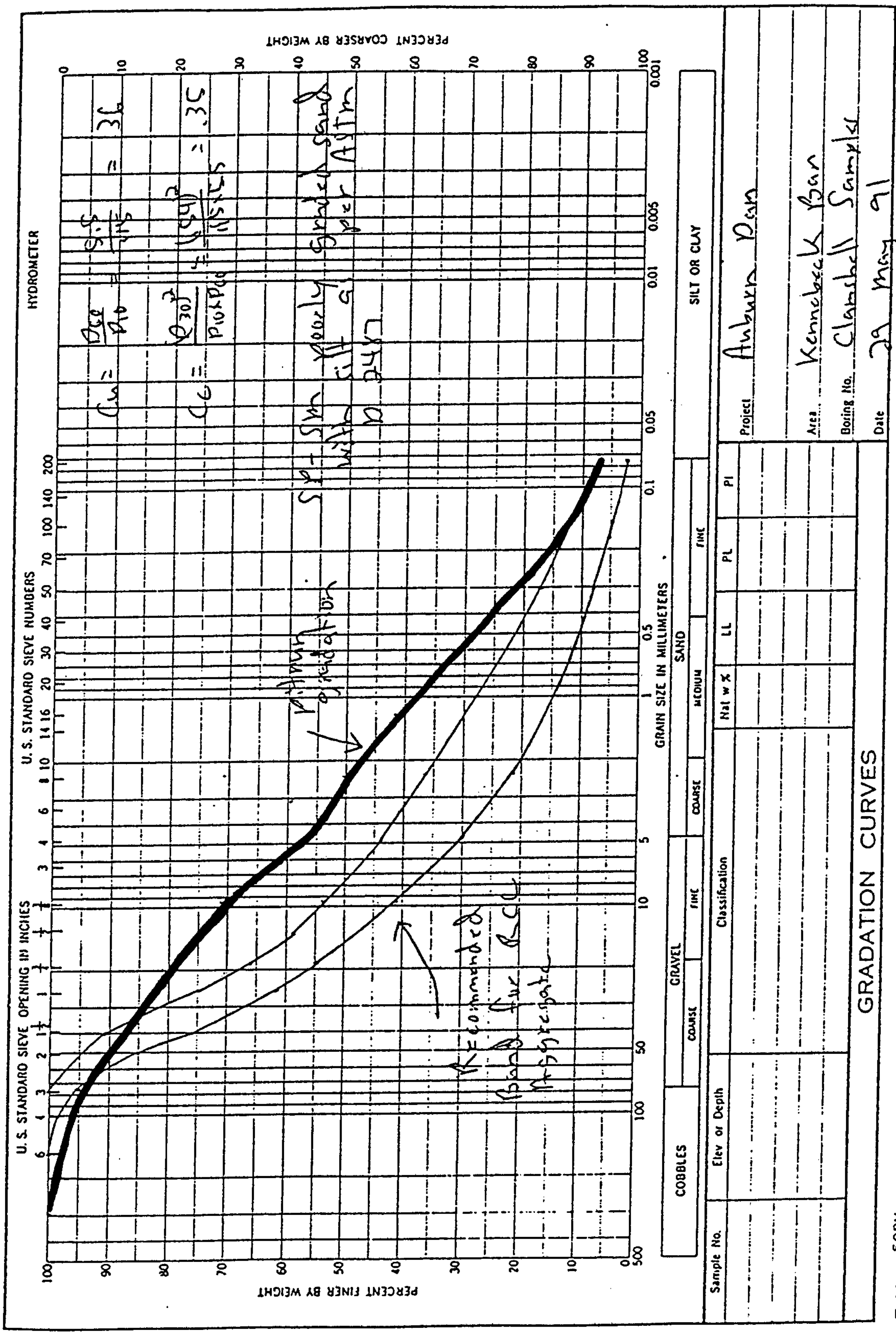
MIDDLE FORK AMERICAN RIVER BARS
MAMMOTH BAR GRADATION
FIGURE 5



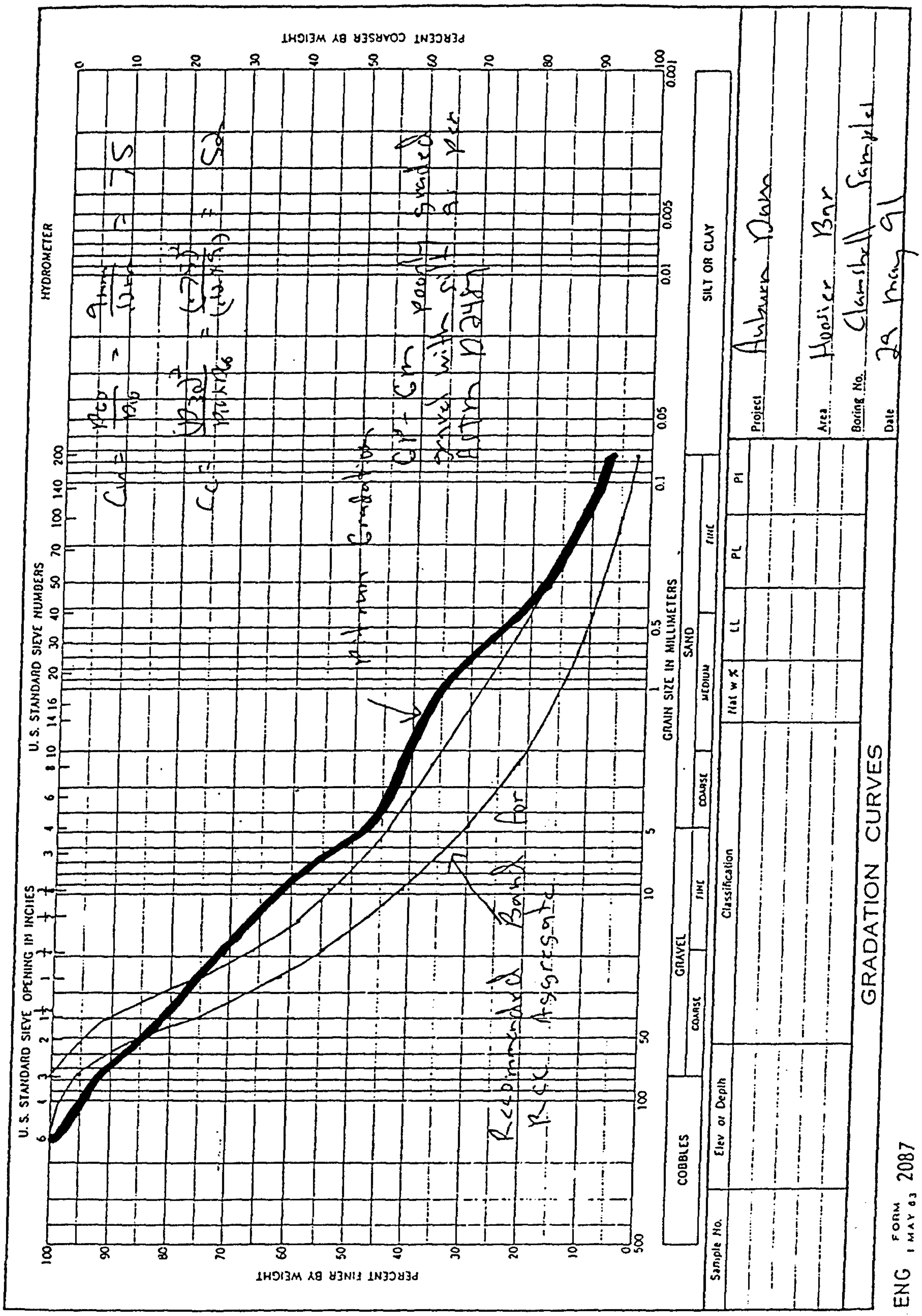
MIDDLE FORK AMERICAN RIVER BARS
TEXAS BAR GRADATION
FIGURE 6



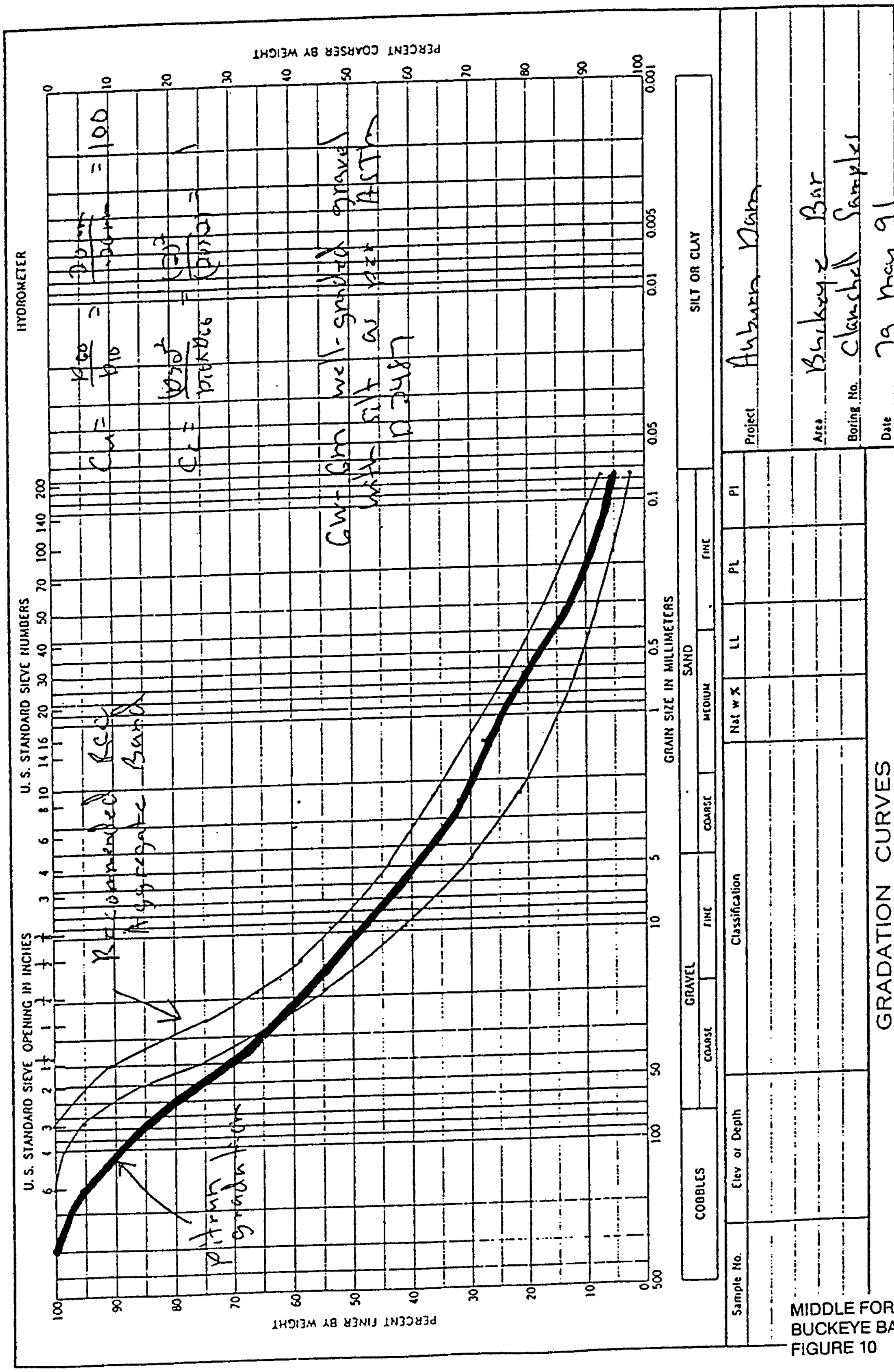
MIDDLE FORK AMERICAN RIVER BARS
BROWNS BAR GRADATION
FIGURE 7



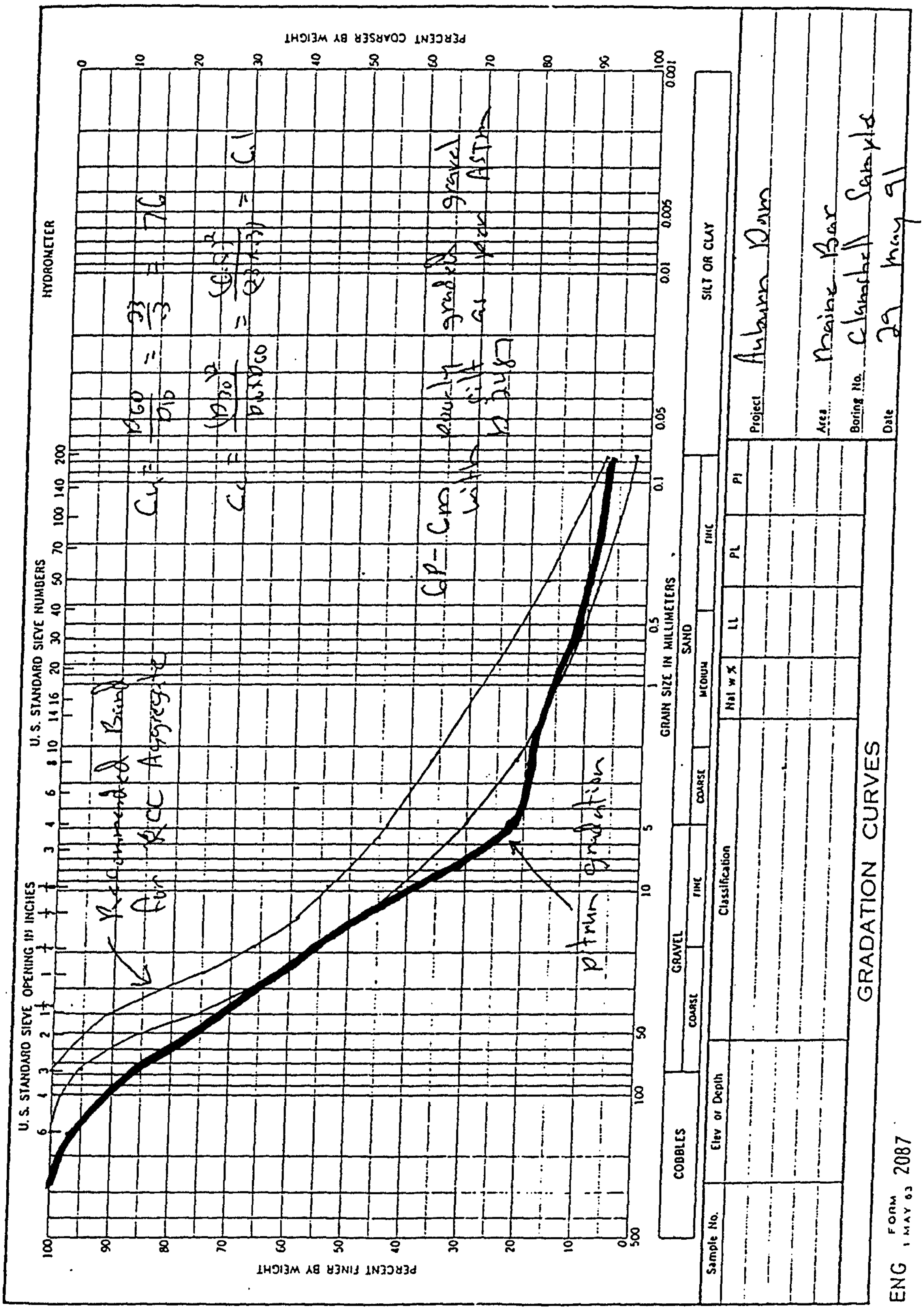
MIDDLE FORK AMERICAN RIVER BARS
KENNEBECK BAR GRADATION
FIGURE 8



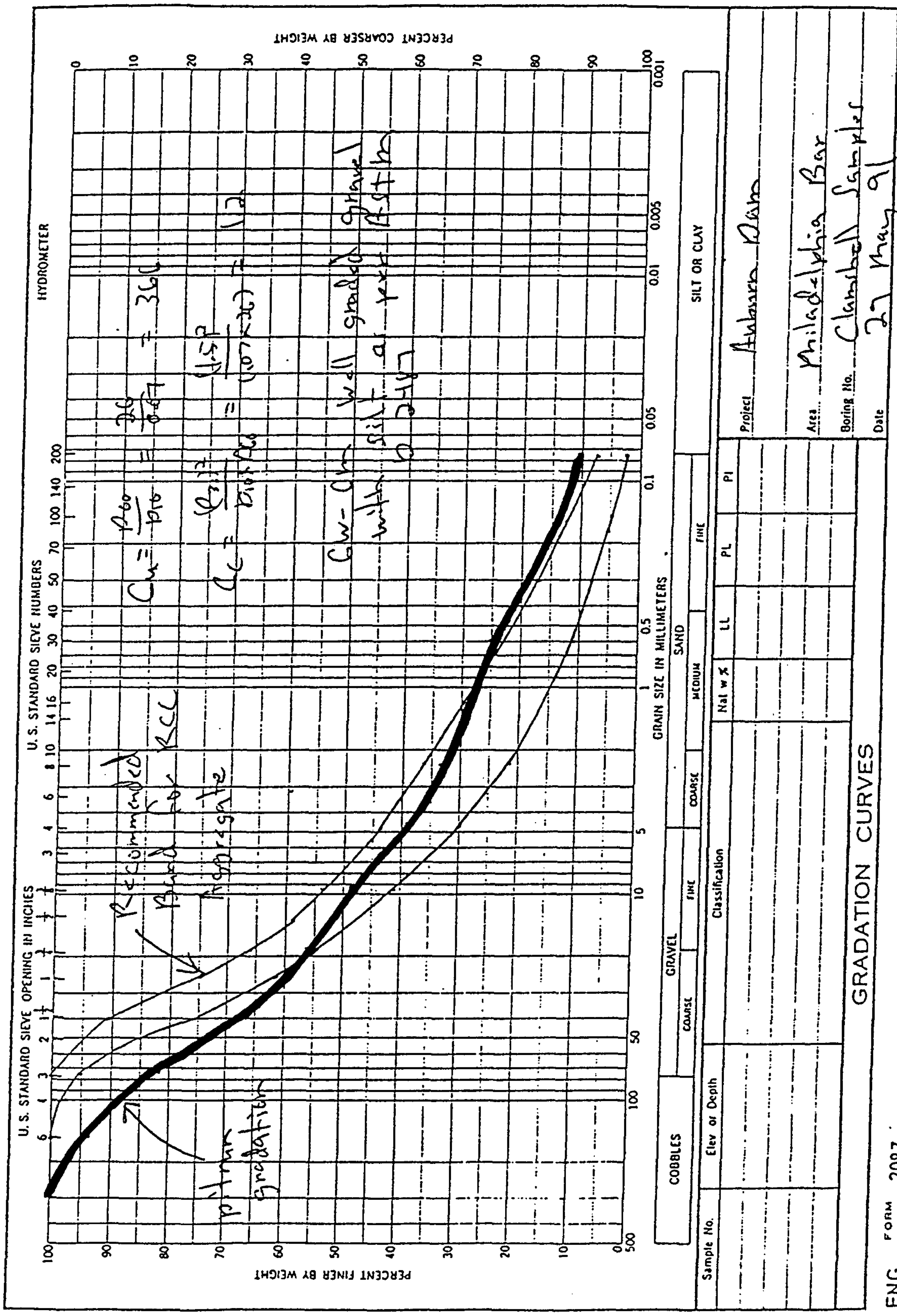
MIDDLE FORK AMERICAN RIVER BARS
HOOSIER BAR GRADATION
FIGURE 9



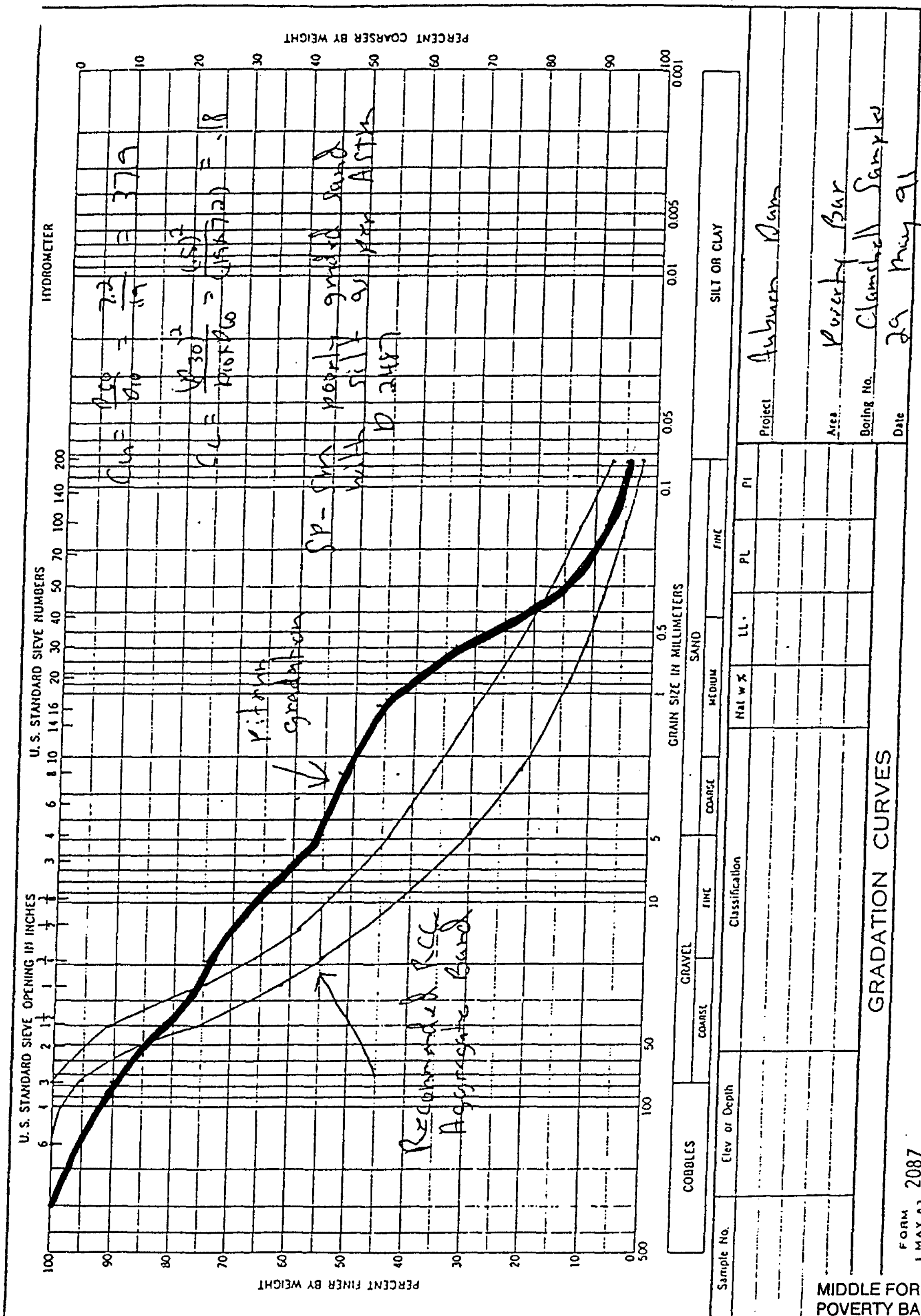
MIDDLE FORK AMERICAN RIVER BARS
BUCKEYE BAR GRADATION
FIGURE 10



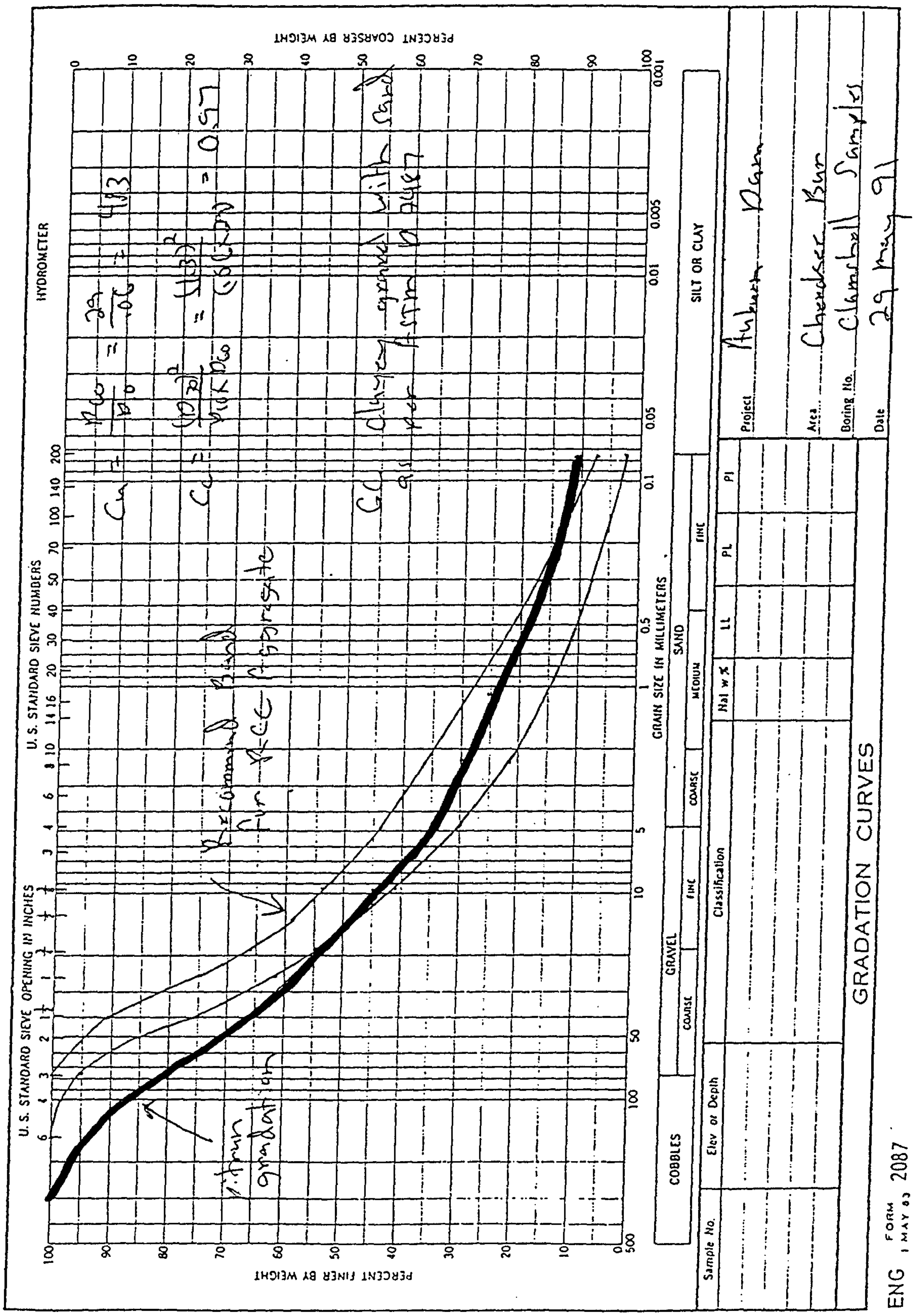
MIDDLE FORK AMERICAN RIVER BARS
MAINE BAR GRADATION
FIGURE 11



MIDDLE FORK AMERICAN RIVER BARS
PHILADELPHIA BAR GRADATION
FIGURE 12



MIDDLE FORK AMERICAN RIVER BARS
POVERTY BAR GRADATION
FIGURE 13

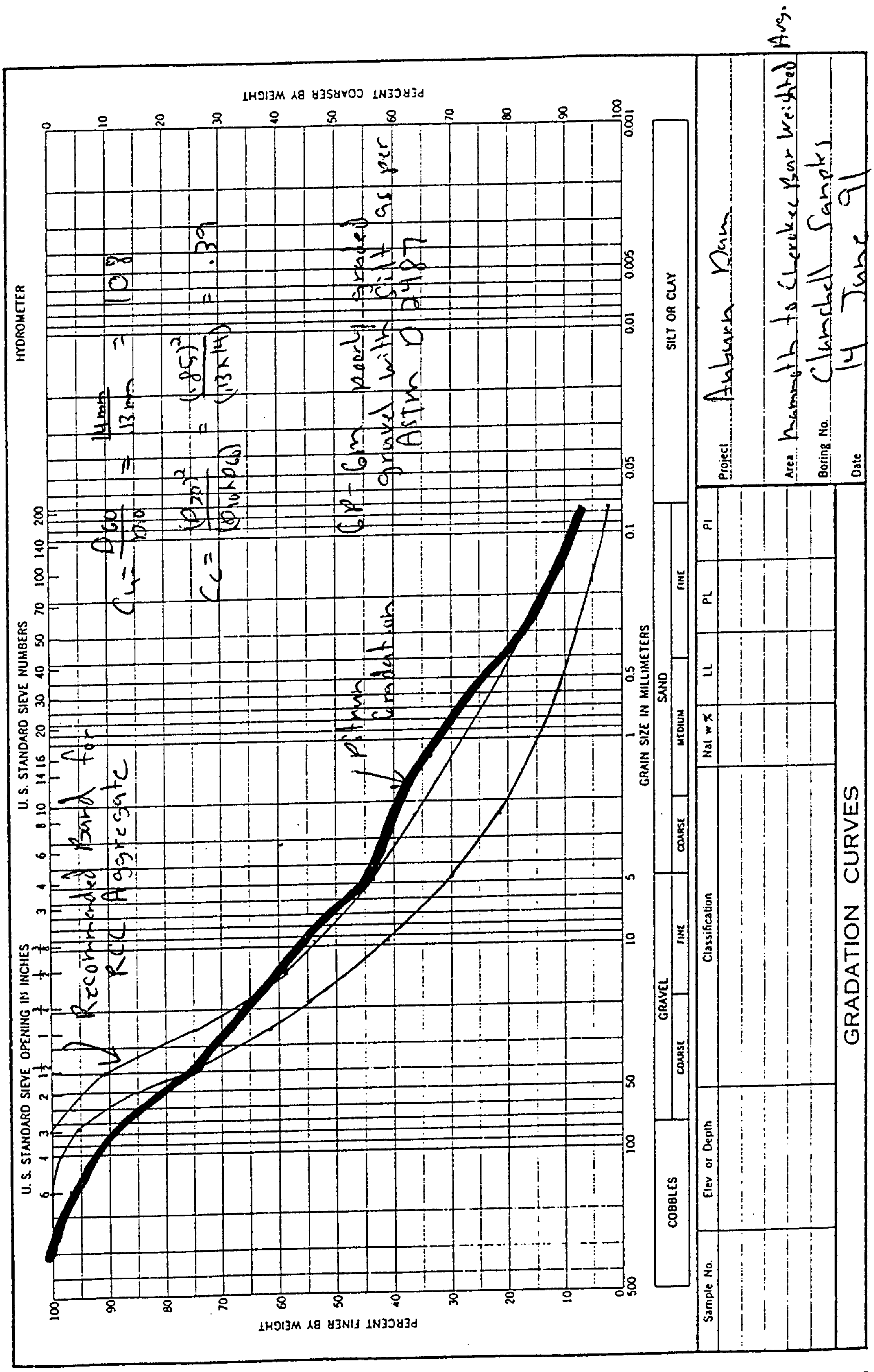


MIDDLE FORK AMERICAN RIVER BARS
CHEROKEE BAR GRADATION
FIGURE 14

LOCATION	AVG. DEPTH TO BEDROCK	AREA (FT)	VOL. (YD)	% >3 in *	% SAND	% FINES
Mammoth Bar	31.4	971,250	1,129,500	14	36	9.5
Texas Bar	31.2	996,000	1,150,900	12	34.5	8.5
Browns Bar	30.4	599,000	674,400	9	36	7.4
Kennebeck Bar	31.5	719,000	839,000	6	51	6
Hoosier Bar	25.9	649,000	622,500	8	40	6.7
Buckeye Bar	27.6	1,104,000	1,128,000	15	32	7.3
Maine Bar	21.0	249,000	194,000	13	15	7.1
Philadelphia Bar	23.7	907,000	705,000	17	28	10.6
Poverty Bar	27.5	1,152,000	1,173,000	10	52	5.8
Cherokee Bar	18.0	1,484,000	989,000	18	22	12.7
Weighted Avg.	26.5'	Total	8,605,000	12%	36%	8.2%

* + 6 in. sized particles were obtained from within 38 in. diameter casing with the clamshell sampler.

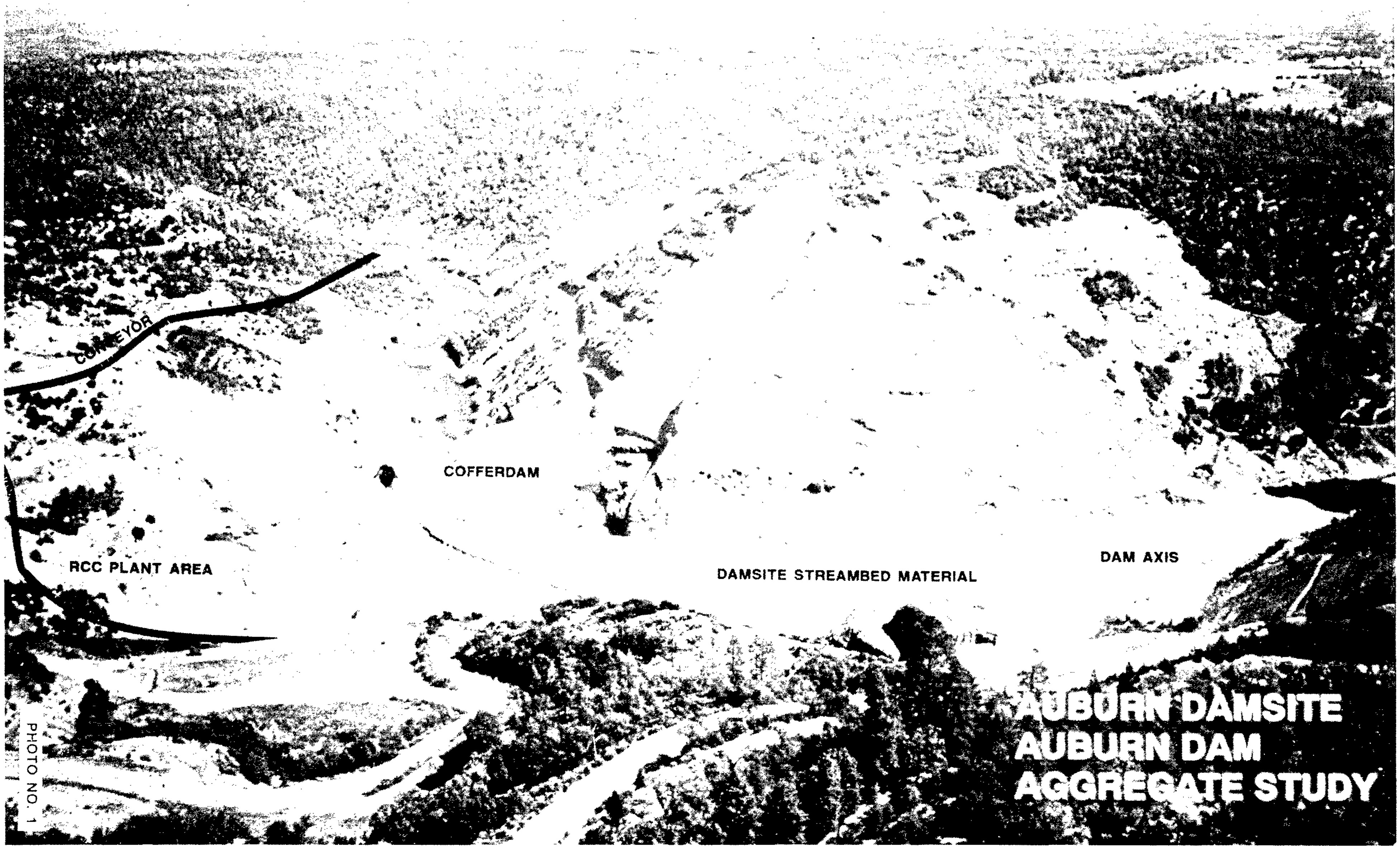
Middle Fork American River
Bar Gradations
Summary Data
Figure 15



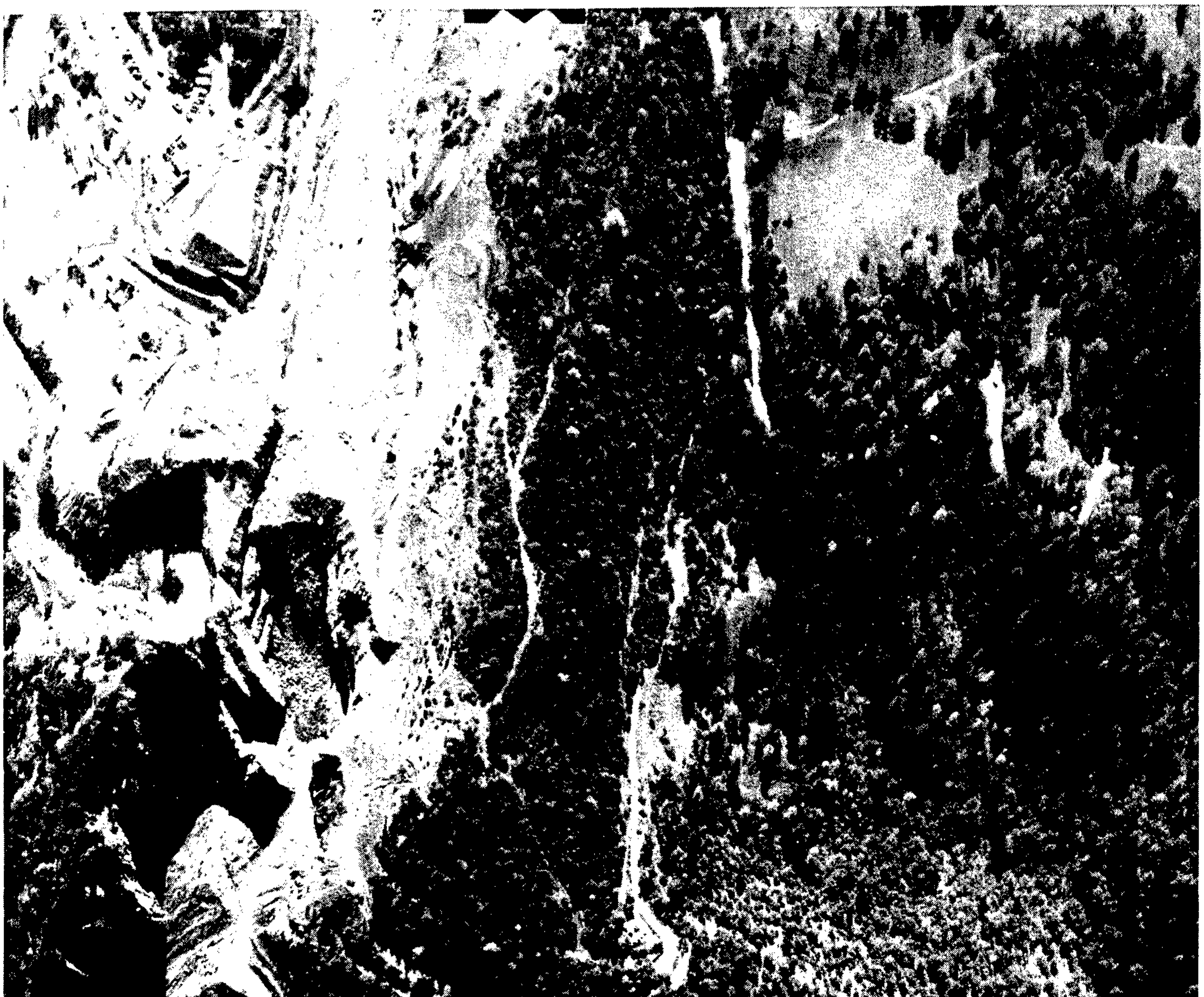
MIDDLE FORK AMERICAN RIVER BARS

OVERALL GRADATION

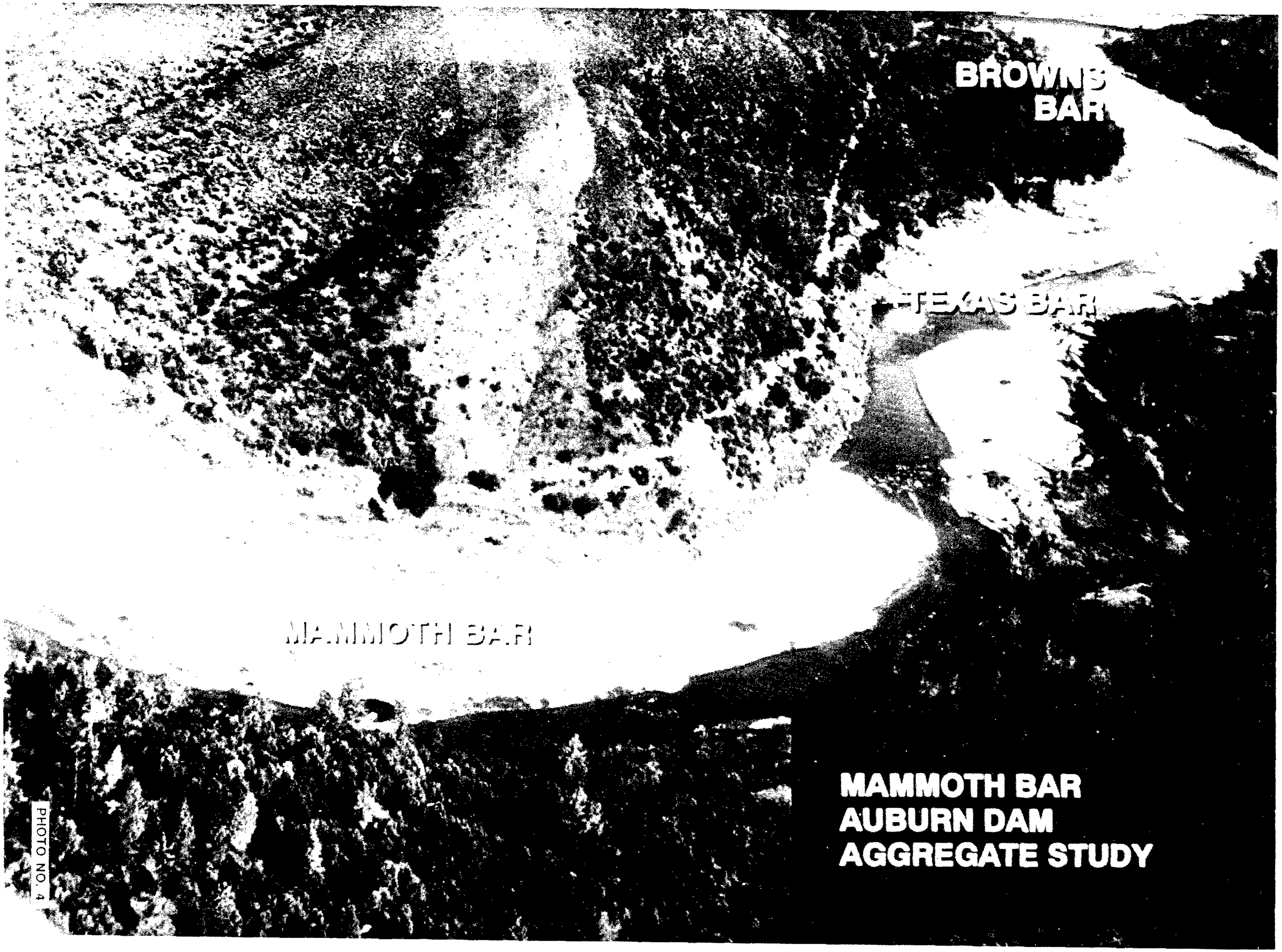
FIGURE 16


AUBURN DAMSITE
AUBURN DAM
AGGREGATE STUDY
CONVEYOR
COFFERDAM
RCC PLANT AREA
DAMSITE STREAMBED MATERIAL
DAM AXIS
PHOTO NO. 1

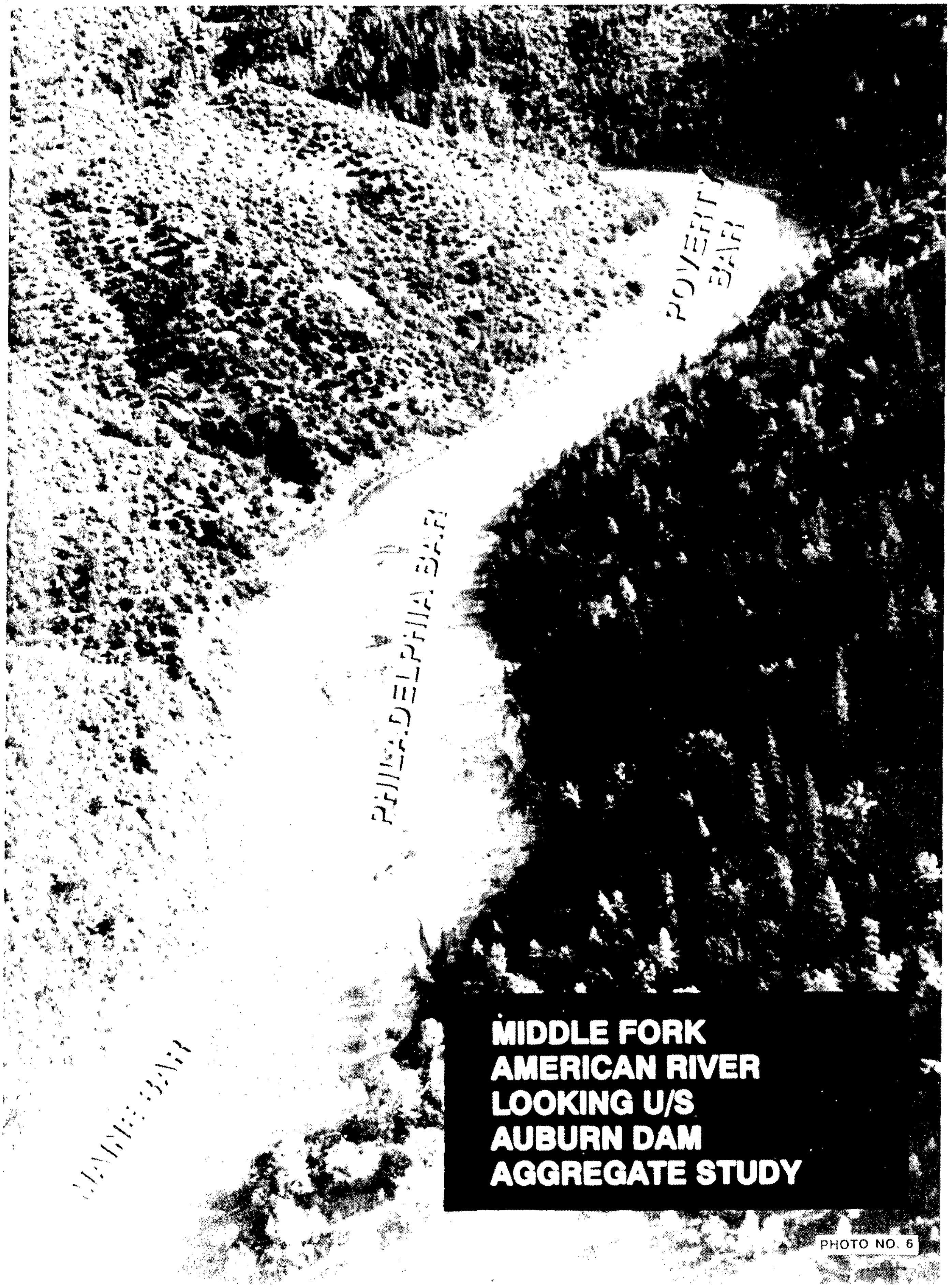

POTENTIAL QUARRY
NEAR COOL QUARRY
AUBURN DAM
AGGREGATE STUDY
PHOTO NO. 2



POTENTIAL AMPHIBOLITE QUARRY
NEAR COOL QUARRY
U/S TO LEFT
AGGREGATE STUDY
AUBURN DAM
PHOTO NO. 3

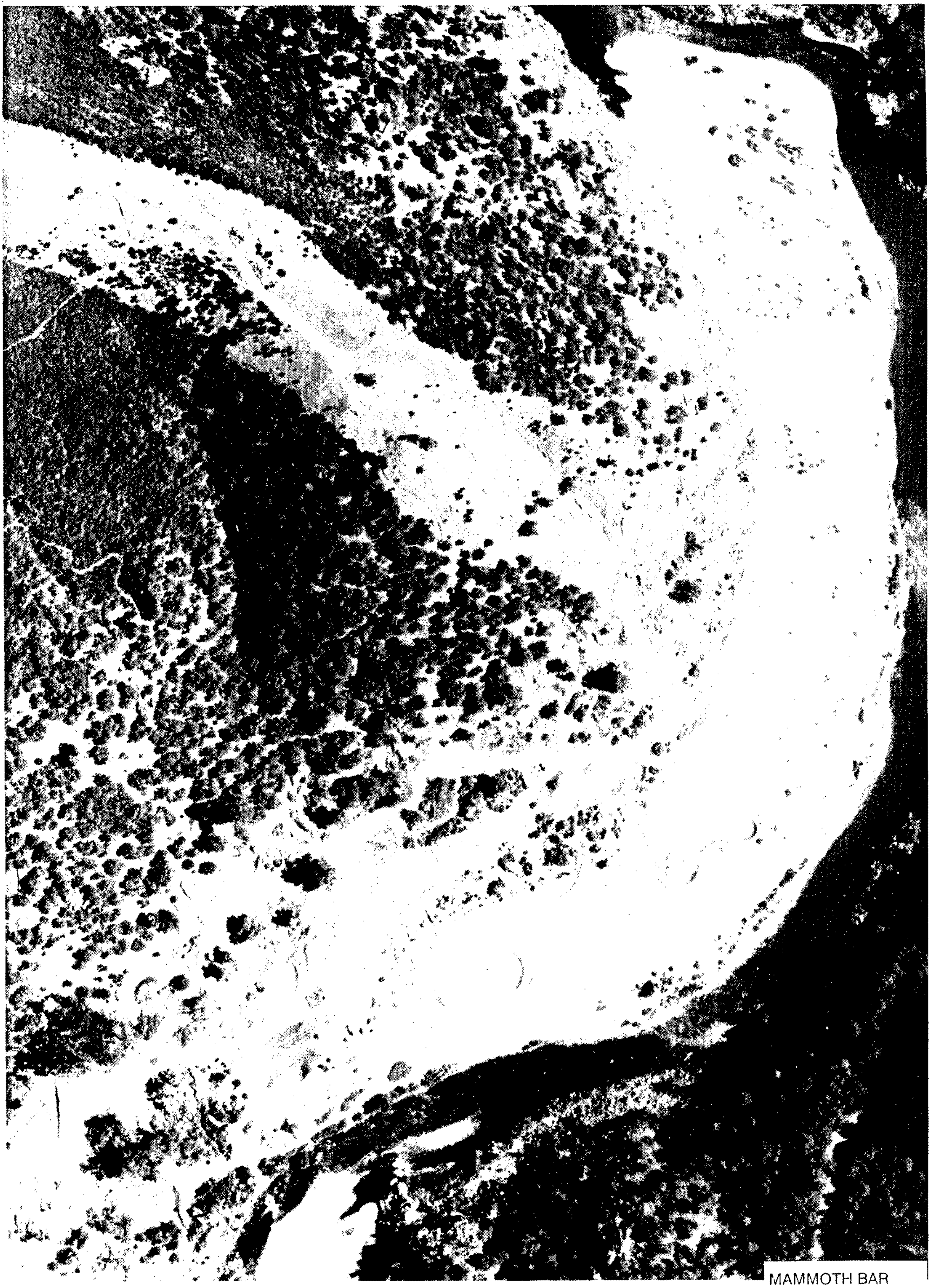

BROWNS
BAR
TEXAS BAR
MAMMOTH BAR
MAMMOTH BAR
AUBURN DAM
AGGREGATE STUDY
PHOTO NO. 4


BUCKEYE BAR
HOOSIER BAR
KENNEBECK BAR
BROWNS BAR
MIDDLE FORK
AMERICAN RIVER
LOOKING U/S
AUBURN DAM
AGGREGATE STUDY
PHOTO NO. 5



MIDDLE FORK
AMERICAN RIVER
LOOKING U/S
AUBURN DAM
AGGREGATE STUDY

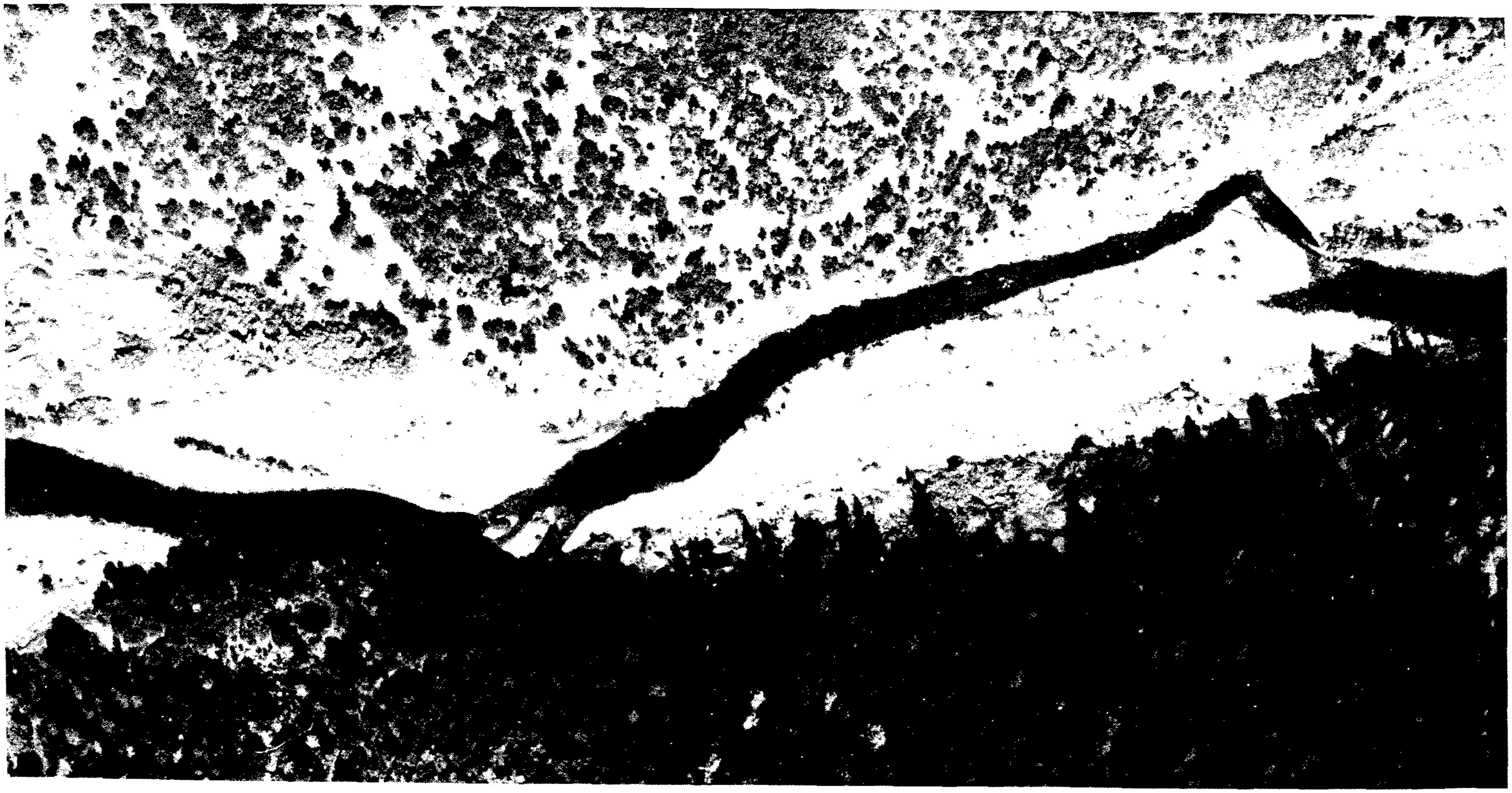
PHOTO NO. 6



MAMMOTH BAR
LOOKING U/S
AGGREGATE STUDY
AUBURN DAM
PHOTO NO. 7

PHOTO NO. 8

BROWNS BAR
LOOKING U/S
AGGREGATE STUDY
AUBURN DAM
PHOTO NO. 9



KENNEBECK BAR
U/S TO RIGHT
AGGREGATE STUDY
AUBURN DAM

PHOTO NO. 10

HOOSIER BAR
U/S TO RIGHT
AGGREGATE STUDY
AUBURN DAM

PHOTO NO. 11

BUCKEYE BAR
U/S TO RIGHT
AGGREGATE STUDY
AUBURN DAM

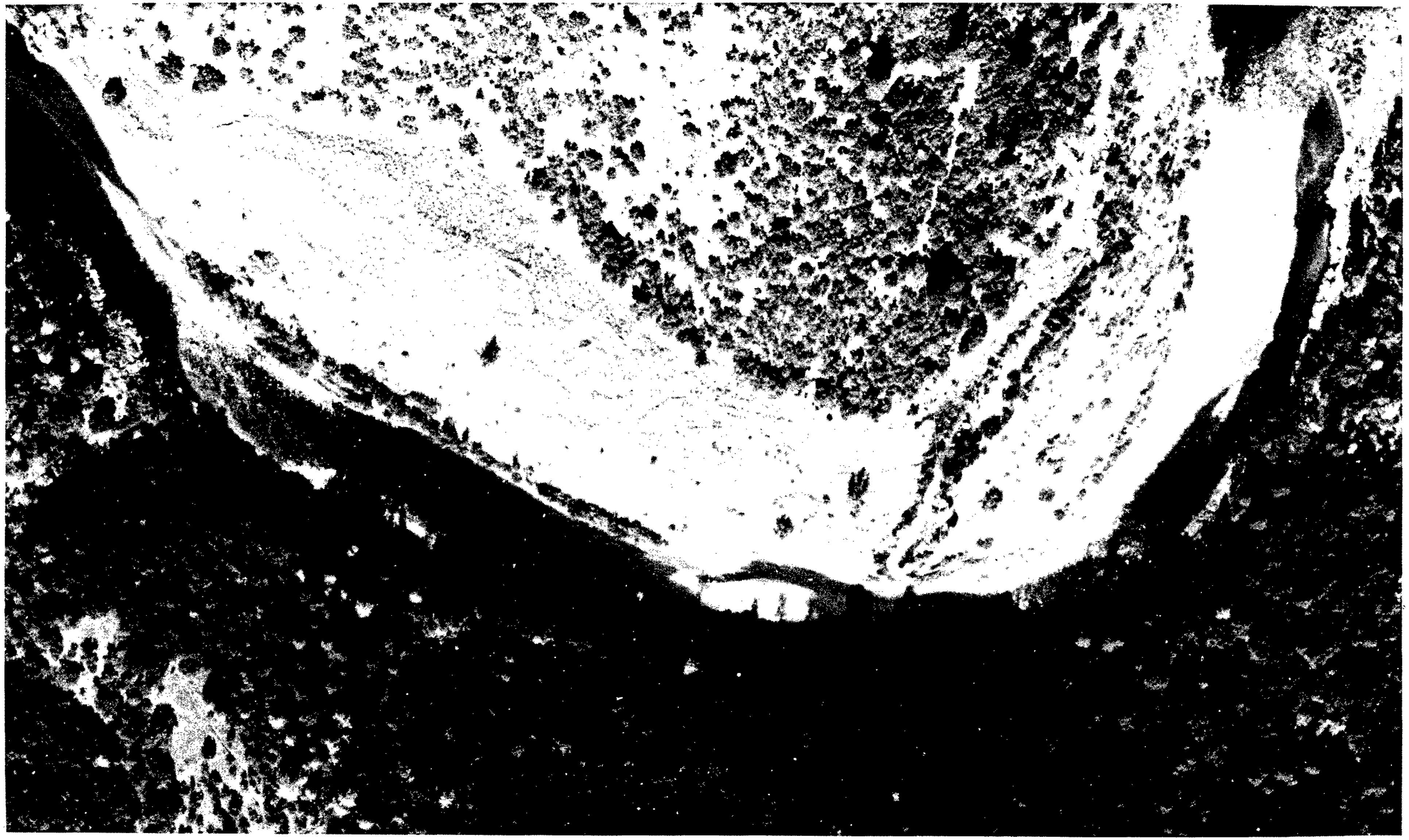
PHOTO NO. 12

MAINE BAR
U/S TO RIGHT
AGGREGATE STUDY
AUBURN DAM

PHOTO NO. 13

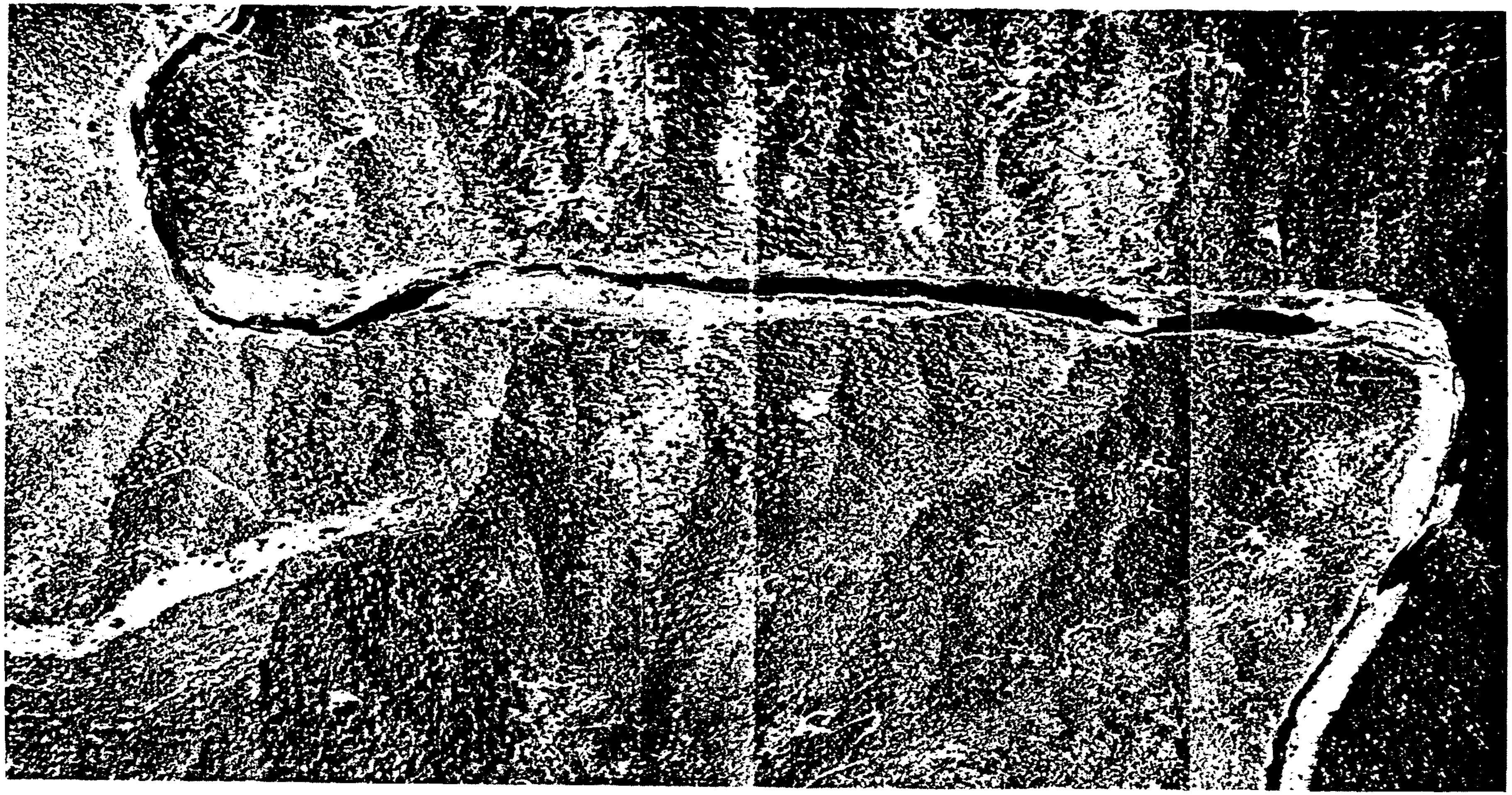
PHILADELPHIA BAR
U/S TO RIGHT
AGGREGATE STUDY
AUBURN DAM

PHOTO NO. 14

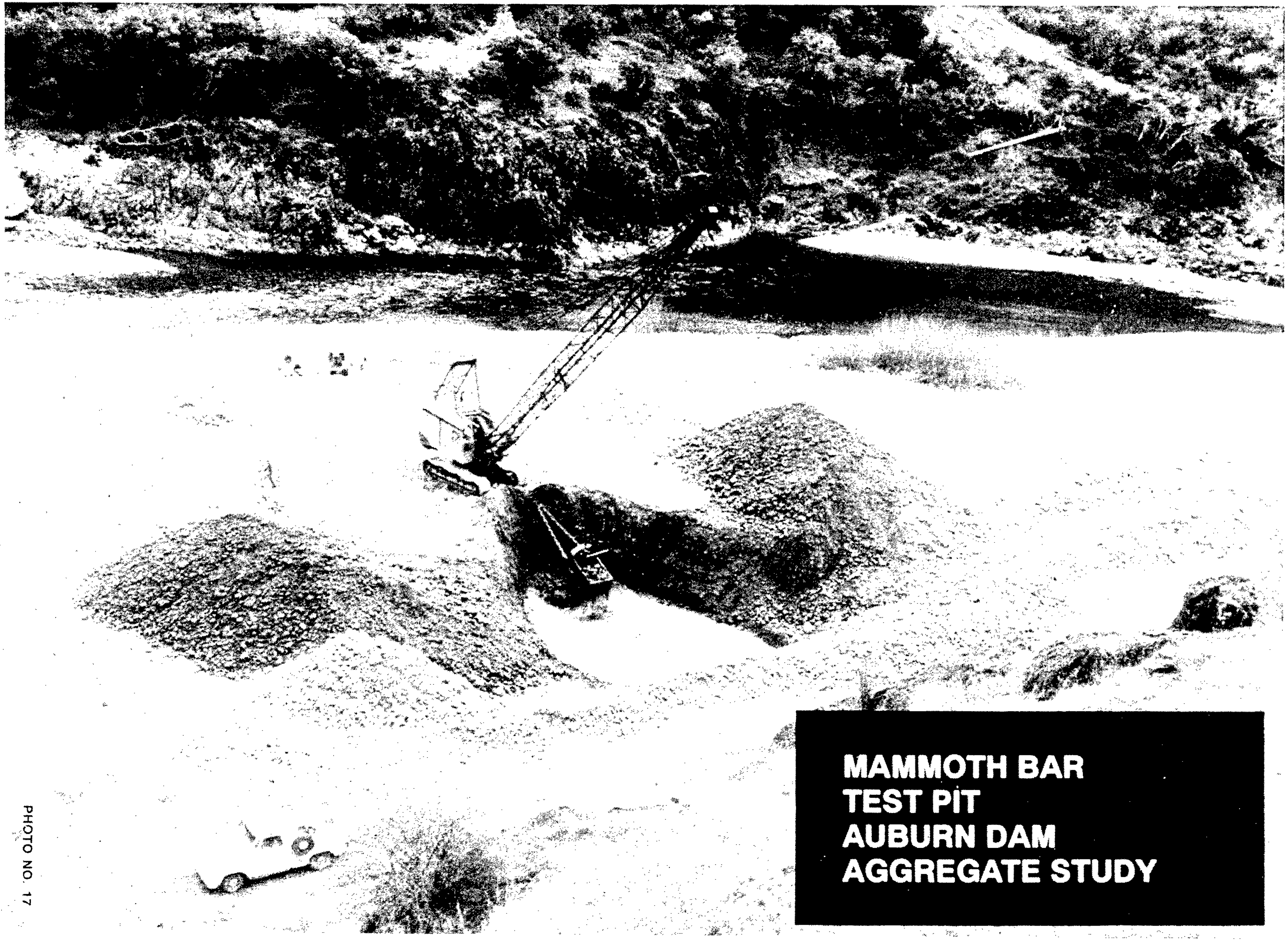


POVERTY BAR
U/S TO RIGHT
AGGREGATE STUDY
AUBURN DAM

PHOTO NO. 15



CHEROKEE BAR
LOOKING U/S
AGGREGATE STUDY
AUBURN DAM
PHOTO NO. 16



MAMMOTH BAR
TEST PIT
AUBURN DAM
AGGREGATE STUDY

PHOTO NO. 17

MAMMOTH BAR
TEST PIT
AUBURN DAM
AGGREGATE STUDY

PHOTO NO. 18

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 10

ENVIRONMENTAL ASSESSMENT OF AGGREGATE SOURCE ALTERNATIVES FOR CONSTRUCTION OF THE PROPOSED FLOOD CONTROL DAM AT AUBURN

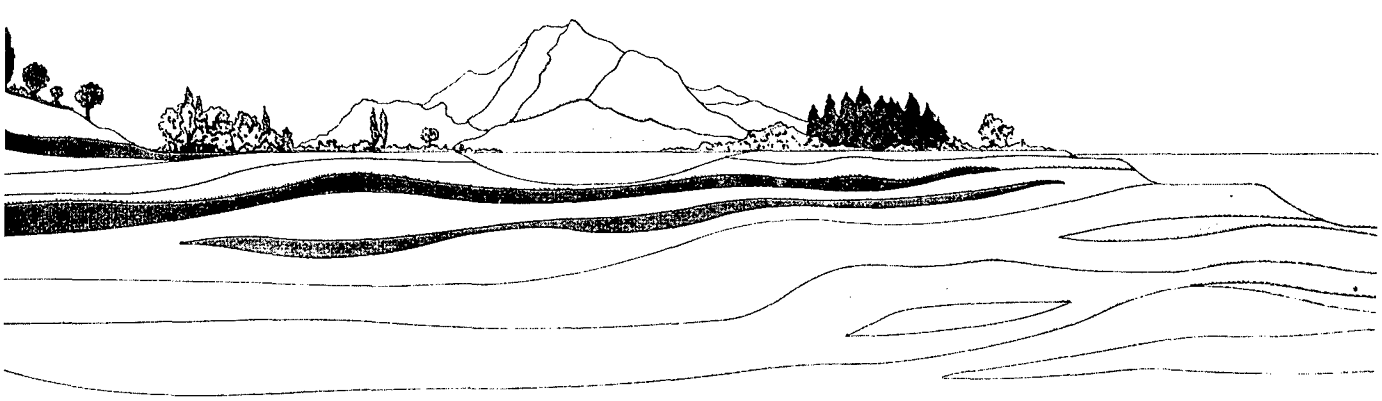
FUGRO McCLELLAND (WEST), INC.

FUGRO-McCLELLAND (WEST), INC.

Environmental Assessment of Aggregate Source Alternatives for Construction of the 200-year Flood Control Dam at Auburn American River Watershed Investigation

prepared for
Department of the Army
U.S. Army Engineer District, Sacramento, CA
and
California Department of Water Resources

November 1991



ENVIRONMENTAL ASSESSMENT OF AGGREGATE SOURCE ALTERNATIVES FOR CONSTRUCTION OF THE 200-YEAR FLOOD CONTROL DAM AT AUBURN AMERICAN RIVER WATERSHED INVESTIGATION

NOVEMBER 1991

Prepared for:
Department of the Army
U.S. Army Engineer District, Sacramento
Sacramento, CA

and
California Department of Water Resources

by
Fugro-McClelland (West), Inc.
1540 River Park Drive Suite 117
Sacramento, CA 95815-4608

ENVIRONMENTAL ASSESSMENT OF
AGGREGATE SOURCE ALTERNATIVES
FOR CONSTRUCTION OF THE
200-YEAR FLOOD CONTROL DAM AT AUBURN
AMERICAN RIVER WATERSHED INVESTIGATION

TABLE OF CONTENTS

ENVIRONMENTAL ASSESSMENT OF
AGGREGATE SOURCE ALTERNATIVES
FOR CONSTRUCTION OF THE
200-YEAR FLOOD CONTROL DAM AT AUBURN
AMERICAN RIVER WATERSHED INVESTIGATION

TABLE OF CONTENTS (CONTINUED)

ENVIRONMENTAL ASSESSMENT OF
AGGREGATE SOURCE ALTERNATIVES
FOR CONSTRUCTION OF THE
200-YEAR FLOOD CONTROL DAM AT AUBURN
AMERICAN RIVER WATERSHED INVESTIGATION

TABLE OF CONTENTS (CONTINUED)

LIST OF FIGURES

LIST OF FIGURES (CONTINUED)

LIST OF TABLES

1.0 INTRODUCTION

One of the Flood Control Alternatives identified in the American River Watershed Investigation (COE, 1991) is construction of a flood control dam capable of containing a flooding event with a 200-year recurrence interval. In order to construct a dam of this size, an estimated 6.76 million cubic yards (9.125 million tons) of aggregate would be required to produce the requisite 5 million yards of concrete. The U.S. Army Corps of Engineers (the Corps), in its June, 1991 Special Aggregate Report, assessed the feasibility of the following aggregate sources in the immediate vicinity of the dam site (Figure 1-1, Regional Location Map):

- **Middle Fork American River Sand and Gravel Deposits** - A series of 10 sand and gravel bars located along a 7-mile reach of the Middle Fork starting approximately 5 miles upstream from the dam site.

- **North Fork American River Sand And Gravel Deposits** - Fluvial sand and gravel deposits located in the backwaters of and beneath Lake Clementine.

- **Old Cool Quarry (Spreckles Limestone and Aggregate)** - A currently operating commercial quarry located in the canyon of the Middle Fork, approximately 5 miles upstream from the dam site.

- **Cool Quarry Amphibolite** - A quarriable body of metamorphic rock located immediately west (downstream) of the existing Cool Quarry (Spreckles).

- **RM 22.4 Quarry** - A quarriable body of metamorphic rock located approximately 2 miles upstream from the dam site.

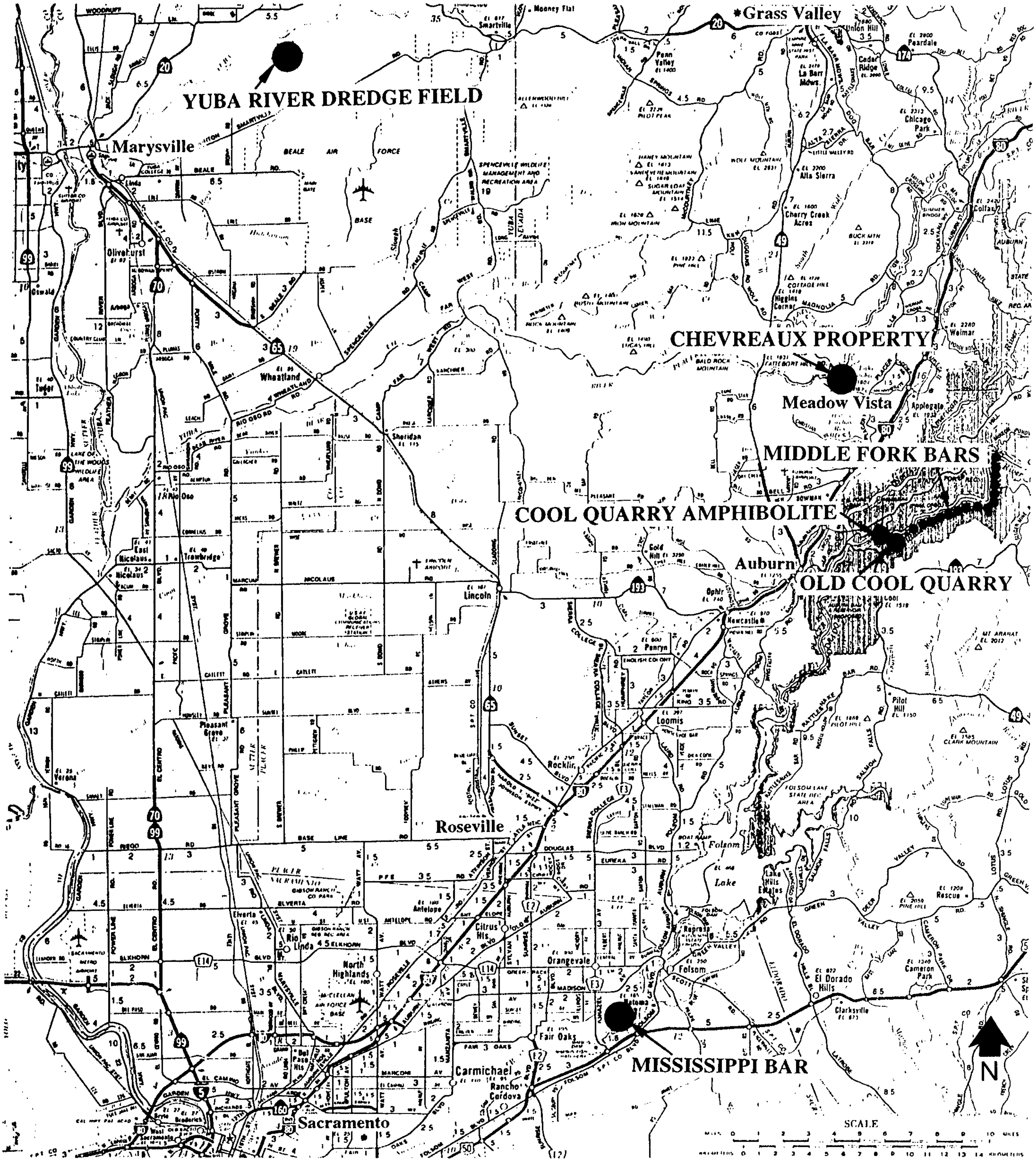
- **Oregon Bar Pluton** - A quarriable body of granitic rock located approximately 1.5 miles downstream of the dam site.

In addition, the Corps reviewed the following sources distant to the dam site (Figure 1-1, Regional Location Map):

- **Mississippi Bar on the Lower American River** - An extensive deposit of sand and gravel located approximately 18 miles from the dam site near Lake Natoma.

- **Bear River and Chevreaux Quarry** - Fluvial deposits of sand and gravel and quarriable rock located on the Bear River, along highway 49, approximately 11 transport miles north of the dam site.

FUGRO McClelland



Regional Location Map

FIGURE 1-1

Yuba River Dredge Fields - Extensive deposits of sand and gravel located approximately 40 miles north of the dam site on the Yuba River north of Beale AFB.

The following sections describe the potential aggregate source locations, and reviews their feasibility as a source of construction material for the dam. Later sections place emphasis on those sources identified in the feasibility assessment as likely candidates for further investigation through development of additional information on mining, transport and processing of aggregate. Information contained in this project description and feasibility assessment was derived from the Corps' Special Aggregate Report (1991), textbooks and industry publications, and discussions with management representatives from the various commercial operators. Environmental impacts associated with utilization of those sources deemed potentially feasible are described in subsequent chapters.

1.1 AGGREGATE SOURCES - OBJECTIVES AND FEASIBILITY

Primary considerations in choosing an aggregate source for the dam include whether a particular source is sufficient in quantity and whether the material satisfies specifications for concrete aggregate. Once it is determined that adequate quantity and quality exists, other factors such as financial and environmental considerations affect a given source's suitability. All of the potential sources listed above contain sufficient reserves of material to serve the purposes of the project (except perhaps for the North Fork deposits for which insufficient information exists to make a determination). However, as noted above, other factors bear on their suitability. The purpose of this section of the report is to preliminarily assess the feasibility and environmental impact potential of the various aggregate sources. Unsuitable sources are eliminated at this stage of the analysis and will not be considered further. Those sources with a demonstrated potential for fulfilling the objectives of an aggregate source (low cost, environmental acceptability and ability to meet design specifications) are examined in detail in later sections of this report.

1.1.1 AGGREGATE SOURCES IN THE DAM SITE VICINITY

Of the six potential sources proximal to the dam site, **Oregon Bar Pluton** can be discounted due to material inadequacies. Due to extensive shearing and deep disintegration of the rock, the Oregon Bar Pluton is not considered a potential source of concrete aggregate.

Quarrying of the amphibolite at river mile 22.4 (**RM 22.4 Quarry**) can be eliminated because of the site's proximity to residential land uses. While the rock appears suitable for aggregate and is in close proximity to the dam site, the quarry site would be in full view of the homes built around Robie Point at the edge of Auburn, a distance of 2,500 feet directly across the river from the site (Figure 1-1, Regional Location Map). Working the quarry for a period of several years in full view of the town, with the expected noise problem, may not be acceptable. Mitigation of the huge scar that would be left in the hillside would be difficult.

There are several sand and gravel bars along the **North Fork of the American River** that lie between the backwaters of Lake Clementine and Ponderosa Bridge, a distance of about 4 miles (Figure 1-1, Regional Location Map). The river distance of this source from the dam site is about 10 miles, as compared to 5 river miles between the dam site and Mammoth Bar. Rough estimates indicate between 2 and 4 million cubic yards of aggregate is available in these bars. No estimate was made as to the percentages of sand and gravel in this total. An extensive exploration program would need to be carried out to further determine the quality and quantity of materials in this source.

Mining the material would require an extensive series of settling ponds and other measures would have to be carried out to minimize turbidity caused by release of fines during excavation. Little room is available upstream from Lake Clementine, which is used extensively for recreation, for such ponds. At present there is no vehicle access to the bars except for Ponderosa Way at the bridge, up over the Forest Hill Divide (some 1,400 feet rise in elevation) to Forest Hill Road, and thence about 16 miles to the dam site. Due to the lengthy distance to the dam site (compared to the Middle Fork bars), as well as environmental problems similar to those associated with mining the sand and gravel along the Middle Fork, it is unlikely that this source of aggregate will be further considered.

Little information is currently available regarding aggregate that may be beneath the waters of **Lake Clementine**. Created by construction of North Fork Dam from 1937 to 1939, the primary purpose of the dam was to store debris produced by hydraulic mining in the upstream reaches of the river. Minimal mining activity has occurred since completion of the project, resulting in only minor amounts of material being deposited in the reservoir. Preliminary estimates using aerial photos indicate that a maximum of 389,000 cubic yards of aggregate was deposited along the river prior to reservoir construction. This material, plus an additional unknown amount deposited in the reservoir by limited hydraulic mining and seasonal flooding, comprise the total amount of material beneath Lake Clementine. However, even if the additional post-reservoir deposition doubled the amount of reserves, the quantity appears insufficient for serious consideration as aggregate for the dam. Extracting this relatively small amount of material from significant depths underwater would be extremely expensive, create an enormous turbidity problem, and be technically difficult to accomplish.

The **Old Cool Quarry**, which is currently operated by Spreckles Limestone and Aggregate, has operated as a commercial limestone quarry since early this century. Spreckles leases the property from the USBR on a biannual basis. The material produced has been used for refining sugar, glass manufacture, cosmetics, roofing material, road base, concrete aggregate, riprap, and various chemical applications. The operator estimates reserves of 12 million tons of marble and 100 million tons of metavolcanic breccia. The currently permitted quarry has a 600 tons per hour (tph) processing capability which could be expanded to 1,000 tph and enough available on-site storage space to stockpile several million cubic yards (Bartley, pers. comm., 1991). Wash water is obtained and discharged on-site and private roads linking the quarry to the dam site exist. Minimal incompatible land uses exist in the

surrounding area. For these reasons, this site is considered to be one of the least environmentally disruptive of the potential aggregate sources and thus requires additional inquiry.

Located immediately west of the existing Cool Quarry (Figure 1-1, Regional Location Map), the potential aggregate source known herein as the **Cool Quarry Amphibolite** has been sampled by the U.S.Bureau of Reclamation (USBR) and petrographically examined for suitability as aggregate material. The results of the petrographic examination were favorable. However, physical testing of this material for use in concrete was never done, nor was an official quarry report published. While environmental concerns exist at this site (see Section 3.0), they are not as significant as those of the RM 22.4 Quarry. The proposed quarry site is adjacent to a large existing quarry (Spreckles), and would be about 2 miles east of Auburn and should be out of direct view. While development of a quarry at this site is environmentally problematic, this site is considered the most likely candidate for a new quarry operation in the vicinity of the dam site; hence, it will be investigated further in this report.

The total estimated quantity of material available from the sand and gravel deposits along the **Middle Fork American River** is about 9.6 million cubic yards. If the river channel materials are ignored, the total quantity of material from the sand and gravel bars alone is 8.6 million cubic yards, or about 27 percent more material than the quantity required for the dam. The results of testing of the bars by USBR indicate the deposits are generally suitable for use as concrete aggregate. However, some washing and crushing of the pitrun (unprocessed) material will be necessary to achieve the requisite particle size gradation.

Because of the investigations performed by the USBR during the 1960s and 1970s, the Middle Fork Bars are the best characterized of the proximal alternative aggregate sources. However, harvest of the deposits is logistically and environmentally problematic (see Section 3.0). Space is restricted along the river. Numerous roads would need to be developed from existing county and state roads to the bars to haul equipment and provide access to conveyors and to the aggregate processing and stockpiling areas. Existing access roads would likely be used as much as possible, but they will need to be improved and widened. Space would need to be made available along the river for stockpiles, vehicle maintenance facilities, and aggregate processing.

The environmental restrictions associated with control of effluent quality during aggregate excavation and processing would require specialized state-of-the-art equipment and treatment methods. Numerous settlement ponds will be needed in the floodplain of the river to trap sediment and allow removal from the flow using flocculating agents. For the aggregate plant(s), a closed-end system will be needed to process and recirculate the wash water. In spite of these restrictions, the close proximity of the Middle Fork bars to the point of use, and the fact that they contain sand and gravel in sufficient quantity and quality to satisfy engineering design considerations, make this source a candidate among sand and

gravel sources in the dam site vicinity. Later sections of this project description will review operational considerations for development of this alternative.

1.1.2 AGGREGATE SOURCES DISTANT TO THE DAM SITE

Of the three distant sources considered in the Special Aggregate Study, the **Bear River and Chevreaux Quarry** near the town of Meadow Vista are the closest sources of large quantities of concrete-grade aggregate (Figure 1-1, Regional Location Map). Historically, a number of operations have extracted sand from the Bear River, of which the largest deposit is owned by the Joe Chevreaux Company. Sand from the Chevreaux operation is obtained from dredging in Lake Combie on the Bear River. Available reserves are estimated to be well in excess of what would be needed for the dam project. In addition to sand, the company obtains coarse aggregate by quarrying and crushing rock located near the river. This material has been used for road base, drain rock, ballast, filter media, road rock, fill material, concrete aggregate, and landscape material (the larger as riprap). The operator estimates reserves of 80 million tons. The rock is classified as metavolcanic breccia. Haul distance to the dam site is approximately 11 miles via public roads.

Originally, USBR considered the Chevreaux property a prime source of aggregate material for the concrete arch dam. Extensive testing of the material was undertaken in the 1960s. However, while the Chevreaux property may have the quality and quantity of material necessary to construct the dam, transport of the aggregate to the dam site is a major environmental obstacle. Because rail transport is not a possibility for this source, material would need to be hauled by large numbers of trucks through residential areas. The owner and operator of the company, Joe Chevreaux, has emphatically stated that he does not want his operation considered as the prime source of aggregate material for the dam project. However, because the nature of the material is well characterized, impacts related to utilization of this source will be evaluated further in this report.

Mississippi Bar is a large bar deposit below Folsom Dam, consisting of sands and gravels which were dredged for their gold content from 1917 to 1949 (Figure 1-1, Regional Location Map). According to the Corps' Special Aggregate Report (1991), an excess of 10,000,000 million cubic yards is available. The federally-owned deposits are generally well-suited for use as RCC aggregate and were used to supply concrete aggregate for the construction of Folsom Dam. These deposits were also considered for use as a source of concrete aggregate for the Auburn Dam by USBR in 1967. The government has been leasing the property to commercial suppliers of concrete aggregate since 1957. Haul distance to the dam site via public roads is approximately 18 miles. Rail transport to the dam site is a possibility.

In 1988, USBR implemented an action to permit controlled removal of dredger tailings from a 160-acre site at Mississippi Bar near the American River for aggregate processing purposes while simultaneously shaping the excavated area into a landform more suitable for vegetation and recreation. The action was approved contingent upon implementation of environmental commitments detailed in an Environmental Assessment prepared for the

project. Among the commitments are restrictions on the time of day that mining and hauling of material can occur.

Teichert Aggregates acquired the option to work the property and currently mines the deposits as well as operates a 300 tons/hr processing facility on an adjacent property owned by the state. In contrast to the estimates contained in the Special Aggregate Report (1991), the Environmental Assessment (EA) completed for the action (USBR, 1988), reported 2.22 million cubic yards of aggregate available for mining. A third estimate obtained by recent communication with Teichert personnel (Johnston, pers. comm., 1991) indicated that permitted reserves are 3-4 million cubic yards. The discrepancies probably stem from differences between the total amount of material contained in the deposit and the amount now designated for mining.

In order for the deposits to be considered a major source of aggregate for the dam project, major revisions to land uses at the bar will need to occur to obtain the requisite quantities. Currently, much of Mississippi Bar is designated for habitat and recreational uses in addition to aggregate mining. Also, significant revisions to mining, processing, transporting and reclamation policies set down in the environmental commitments attached to the mining project would be necessary.

The aggregate reserves along the **Yuba River** near Marysville consist of extensive deposits of dredger tailings. Such deposits are similar to those along the American River and at Mississippi Bar, but much larger in total volume. Most of these deposits are government owned, although this ownership is sometimes disputed. Three companies have holdings in the area: Western Aggregate, Baldwin Construction, and Teichert Aggregate. All three have processing facilities in the area. Western's operation is located on the south side of the river and reserves are estimated to be in excess of 3 billion tons (Clausen, pers. comm., 1991); Baldwin's is in excess of 10 million cubic yards (COE, 1991). The quality of the aggregate in this area is well established, having been investigated for the Marysville Dam and Cache Creek projects. The area has been designated as a mineral resource area (CDMG, 1988) and minimal land use conflicts exist.

As with the Chevreaux source, the main obstacles to overcome with the Yuba River deposits are transportation-related. The material could be trucked to a concrete plant near the Auburn dam site, a distance of approximately 40 miles. However, the logistics of this truck transport are truly formidable. To place 5 million yards of concrete within 2 years, approximately 700 to 900 truckloads of aggregate would have to be delivered each day, or about 40 truckloads per hour. The ability of the existing state and county roads to withstand this kind of use is questionable. For this reason, rail transport from the Yuba River fields is a necessary consideration. Western Aggregate anticipates having rail transport available to the Sacramento area in the next several years. If rail transport does become available, the Yuba River sand and gravel deposits are the least environmentally problematic of the aggregate sources distant to the dam site. These environmental considerations are described in later sections of this report.

1.1.3 FEASIBILITY SUMMARY

Of the 9 potential sources discussed in the Special Aggregate Report, **Oregon Bar Pluton** can be eliminated from serious consideration on the basis of material inadequacies. Quarrying of the amphibolite at the **RM 22.4 Quarry** can be discounted because of the site's proximity to adjacent residential land uses. Potential environmental impacts associated with this site include noise and aesthetics.

Utilization of the materials along the **North Fork of the American River** both under and in the backwaters of Lake Clementine is problematic for several reasons. Little information exists regarding the quantity and nature of the deposits, access to the deposits is poor and mitigation of potential water quality impacts associated with extraction of the material would be extremely difficult.

The **Old Cool Quarry** is a viable candidate, particularly from an environmental standpoint. The operation is permitted and currently operating. The material produced has been used as concrete aggregate in the past. Sufficient reserves of material exist and room for stockpiling of finished product is readily available on-site. Water for the existing aggregate processing operation is from an on-site source. Access to the dam site could be had without impacting county and or state roads using conveyors or off-highway trucks.

The **Cool Quarry Amphibolite**, identified by the Corps as a potential new quarry site, would have the same dam site access capabilities as the Old Cool Quarry. Also, the Cool Quarry Amphibolite, like the Old Cool Quarry, has minimal surrounding land use compatibility issues. However, no on-site areas currently exist for aggregate processing or storage. Large areas would have to be set aside for topsoil and overburden storage. Although water for processing aggregate would likely be recycled, provisions would have to be made for acquisition and treatment of the process water. This source is a definite possibility; however, additional environmental and engineering analysis is necessary to further determine feasibility.

The suitability of the sand and gravel deposits along the **Middle Fork American River** for construction material has been ascertained. Preliminary estimates indicate the deposits contain sufficient quantities of concrete-grade aggregate. Extraction, processing and transport of the aggregate is environmentally problematic. Salient issues include turbidity control, waste material disposal, transport and processing site remediation, and geomorphic impacts related to bar removal. Despite these negative aspects, these aggregate deposits are among the most likely candidates for further consideration. This is because of their close proximity to the dam site and the fact that the suitability of the deposits for dam construction is established.

The primary issues plaguing the **Chevreaux and Mississippi Bar** sources are transportation and land use restrictions. Incompatible adjacent land uses at the mining and processing sites as well as along potential haul routes make these areas environmentally problematic.

Preliminary review of the **Yuba River** fields indicate no such restriction exists at the mining and processing site. Because of the possibility for rail transport, transportation impacts, while substantial, may be the easiest to mitigate for these deposits.

Based on review of the Corps' Special Aggregate Study and preliminary environmental review of the potential sources identified in the study, the following six potential aggregate sources will undergo additional environmental review:

- Middle Fork American River Sand and Gravel Deposits
- Old Cool Quarry (Spreckles)
- Cool Quarry Amphibolite
- Chevreaux Property
- Mississippi Bar - American River
- Yuba River Dredge Fields

Of the six possible sources, two (Middle Fork sand and gravel deposits and the Cool Quarry Amphibolite) would require development of new mining operations. The remaining four would entail expansion of existing operations. Primary recommendations in environmental impact minimization is concentration of mining activities in areas (such as the Old Cool Quarry and Yuba River Dredge Fields) already subject to aggregate resource development, and not subject to potential land use conflicts.

2.0 EXECUTIVE SUMMARY

This Environmental Assessment (EA) of alternative aggregate sources for the 200-year flood control dam at Auburn has been prepared for the U.S. Army Corps of Engineers and the California Department of Water Resources. Its purpose is to identify potentially significant environmental impacts associated with the six selected alternatives and preliminarily assess available mitigation measures. The selected alternatives are:

- Middle Fork Sand and Gravel Deposits;
- Old Cool Quarry (Spreckles);
- Cool Quarry Amphibolite;
- Bear River and Chevreaux Quarry;
- Mississippi Bar; and
- Yuba River Dredge Fields.

Environmental issues analyzed for each of these alternatives are:

- Land Uses;
- Public Health and Safety;
- Water Quality;
- Air Quality;
- Biological Resources;
- Transportation;
- Noise;
- Recreation; and
- Visual Resources.

In addition, geomorphic impacts associated with sand and gravel extraction from the floodplain of the Middle Fork of the American River have been assessed. Impacts identified as part of the assessment have been classified according to various administrative actions necessary (see Section 4.0 for a discussion) for project approval. Impacts associated with the individual issue areas are tabulated in Table 2-1 and are summarized below:

Middle Fork Sand and Gravel Deposits. Implementation of this alternative would result in significant unavoidable adverse impacts in the following categories: biological resources, air quality, noise, recreation, visual resources and stream channel morphology. Significant but mitigable impacts would occur in the areas of public health and safety, water quality, biological resources, transportation, and noise. The large amounts of disturbance necessary to fully implement this alternative, coupled with its close proximity to sensitive biological and recreational resources, ranks the Middle Fork Sand and Gravel Deposits as the alternative with the greatest potential for significant environmental impairment.

Old Cool Quarry (Spreckles). In contrast, utilization of the Old Cool Quarry would have the least potential for environmental impairment. This is due in large part to the long history of mining activity at the site. The high level of existing disturbance at the site, its location out of the sensitive canyon bottom, and the possibility of an overland conveyor route to the dam site minimizes its potential for environmental impact. Of the various issue areas, the additional noise generated by an expanded quarry operation is considered significant and unavoidable. Impacts associated with land use, public health and safety, water quality, air quality, transportation, recreation and visual resources were deemed potentially significant, but mitigable. Potential impacts to biological resources at the site were considered insignificant.

Cool Quarry Amphibolite. Because implementation of this alternative would require establishment of a new quarry in a relatively undisturbed area, the potential for significant impact is generally greater when compared to the Old Cool Quarry. Significant unavoidable adverse impacts could occur in the areas of visual resources and noise. Like the Old Cool Quarry, significant but mitigable impacts could occur in the areas of land use, public health and safety, water quality, air quality, transportation, recreation and visual resources. Biological resources could also be significantly impacted.

Bear River/Chevreaux Quarry. Significant unavoidable impacts were identified in the areas of land use, public health and safety, air quality, transportation and noise. These types of impacts result primarily from the large trucking effort required to implement this alternative. Site-specific impacts were found to be potentially significant but mitigable. They include water quality, biological resources, and noise. Impacts to visual resources were considered insignificant.

Mississippi Bar. Significant unavoidable impacts associated with this alternative stem from its location in an urban area and the large scale of the trucking operation. Significant unavoidable impacts were found in the areas of land use, public health and safety, air quality, transportation and noise. Significant but mitigable impacts were indicated in the areas of water quality, biological resources, recreation and visual resources.

Yuba River. The existing aggregate operations, their remote location and the disturbed nature of the area make the Yuba River alternative the least environmentally detrimental of the sources distant to the dam site. As with the other two distant sites, trucking the aggregate from this site would result in significant unavoidable transportation, air quality, noise and public health and safety impacts. Trucking the material 40 miles to the dam site has severe environmental implications. Utilization of rail transport would mitigate some transportation related impacts. Impacts related to other issues such as land use, biological, visual and recreational resources, as well as water quality, are considered potentially significant but mitigable.

TABLE 2-1. ALTERNATIVE AGGREGATE SOURCES IMPACT SUMMARY TABLE

CLASS I. SIGNIFICANT UNAVOIDABLE ADVERSE ENVIRONMENTAL IMPACTS of the project for which the decision-maker must issue a "statement of overriding considerations" under Section 15093 of the State CEQA Guidelines (as amended) if the project is approved.

Impact Summary	Middle Fork Sand and Gravel Deposits	Old Cool Quarry	Cool Quarry Amphibolite	Bear River/ Chevreaux	Mississippi Bar	Yuba River
LAND USE	Impact: See Section 4.7, Noise; 4.8, Recreation; and 4.9, Visual Resources.	Impact: See Section 4.7, Noise, and 4.9, Visual Resources.	Impact: See Section 4.7, Noise, and 4.9, Visual Resources.	Impact: No Class I land use impacts have been identified.	Impact: Expanded operations would not be compatible with surrounding residential and recreational land uses.	Impact: No Class I land use impacts have been identified.
	Impact: Alteration of floodplain morphology would alter recreational use of the river (refer to Section 4.8, Recreation, and Section 4.10, In-Stream Impacts).					
PUBLIC HEALTH AND SAFETY	Impact: No Class I impacts to public health and safety have been identified.	Impact: No Class I impacts to public health and safety have been identified.	Impact: No Class I impacts to public health and safety have been identified.	Impact: 700 to 900 truck deliveries would occur each day through residential and commercial areas.	Impact: 700 to 900 truck deliveries would occur each day through residential and commercial areas.	Impact: 700 to 900 truck deliveries would occur each day through residential and commercial areas.
WATER QUALITY	No Class I impacts to Water Quality have been identified for any of the six alternatives.					

TABLE 2-1. ALTERNATIVE AGGREGATE SOURCES IMPACT SUMMARY TABLE

CLASS I. (CONTINUED).

Impact Summary	Middle Fork Sand and Gravel Deposits	Old Cool Quarry	Cool Quarry Amphibolite	Bear River/ Chevreaux	Mississippi Bar	Yuba River
AIR QUALITY	**Impact:** Due to air basin non-attainment, additional project-related emissions will result in significant unavoidable air quality impacts.	**Impact:** Same as described for Middle Fork Sand and Gravel alternative.	**Impact:** Same as described for Middle Fork Sand and Gravel alternative.	**Impact:** Same as described for Middle Fork Sand and Gravel alternative.	**Impact:** Same as described for Middle Fork Sand and Gravel alternative.	**Impact:** Same as described for Middle Fork Sand and Gravel alternative.
FISH, VEGETATION, AND WILDLIFE	**Impact:** Loss of approximately 34 acres of riparian shrub/scrub, which is also considered wetland under CDFG guidelines.	**Impact:** No Class I impacts to fish, vegetation and wildlife have been identified.	**Impact:** No Class I impacts to fish, vegetation and wildlife have been identified.	**Impact:** No Class I impacts to fish, vegetation and wildlife have been identified.	**Impact:** No Class I impacts to fish, vegetation and wildlife have been identified.	**Impact:** No Class I impacts to fish, vegetation and wildlife have been identified.
TRANSPORTATION	**Impact:** No Class I transportation impacts have been identified.	**Impact:** No Class I transportation impacts have been identified.	**Impact:** No Class I transportation impacts have been identified.	**Impact:** Truck transport of aggregate (800 deliveries per day on public roadways) would reduce the useful life of all affected roadbeds.	**Impact:** Same as described for Bear River/Chevreaux alternative.	**Impact:** Same as described for Bear River/Chevreaux alternative.
				Impact: Increased truck traffic would significantly deteriorate level of service along the haul route.	**Impact:** Same as described for Bear River/Chevreaux alternative.	**Impact:** Same as described for Bear River/Chevreaux alternative.

TABLE 2-1. ALTERNATIVE AGGREGATE SOURCES IMPACT SUMMARY TABLE

CLASS I. (CONTINUED).

Impact Summary	Middle Fork Sand and Gravel Deposits	Old Cool Quarry	Cool Quarry Amphibolite	Bear River/ Chevreaux	Mississippi Bar	Yuba River
NOISE	Impact: Noise from mining and construction activities, particularly earth-moving equipment. Also noise generated from continuous mining, processing, and transporting equipment.	Impact: Same as described for Middle Fork Sand and Gravel alternative.	Impact: Same as described for Middle Fork Sand and Gravel alternative.	Impact: Same as described for Middle Fork Sand and Gravel alternative.	Impact: Same as described for Middle Fork Sand and Gravel alternative.	Impact: Same as described for Middle Fork Sand and Gravel alternative.
				Impact: Increased noise levels generated from transporting material to the dam site through areas with sensitive receptors.	Impact: Same as described for Bear River/Chevreaux alternative.	Impact: Same as described for Bear River/Chevreaux alternative.
RECREATION	Impact: Cumulative loss of the 10 Middle Fork bars would impact all recreational activity within the Middle Fork on a short-term basis (2 to 3 years), and impact several recreational activities permanently.	Impact: No Class I impacts to recreation have been identified.	Impact: Excavation of overburden, large cuts in the slope, and mining activity would visually impact the canyon.	Impact: No Class I impacts to recreation have been identified.	Impact: No Class I impacts to recreation have been identified.	Impact: No Class I impacts to recreation have been identified.

TABLE 2-1. ALTERNATIVE AGGREGATE SOURCES IMPACT SUMMARY TABLE

CLASS I. (CONTINUED).

Impact Summary	Middle Fork Sand and Gravel Deposits	Old Cool Quarry	Cool Quarry Amphibolite	Bear River/ Chevreaux	Mississippi Bar	Yuba River
VISUAL RESOURCES	Impact: Excavation of sand and gravel deposits along a 7-mile stretch of the Middle Fork and excavation for the 12-mile conveyor alignment in the canyon bottom would permanently alter the visual character of this area.	Impact: No Class I impacts to visual resources have been identified.	Impact: Excavation of overburden, large cuts in the slope, and mining activity would visually impact the canyon in this vicinity. In addition, this operation would be adjacent to the existing Old Cool Quarry, enlarging the mining scar which overlooks the canyon.	Impact: No Class I impacts to visual resources have been identified.	Impact: No Class I impacts to visual resources have been identified.	Impact: No Class I impacts to visual resources have been identified.
IN-STREAM IMPACTS (only for Middle Fork Sand and Gravel Deposits Alternative)	Impact: Significant changes in stream morphology. Changes in stream morphology would have significant impacts for recreation, visual, and biological resources.					

TABLE 2-1. ALTERNATIVE AGGREGATE SOURCES IMPACT SUMMARY TABLE

CLASS II. SIGNIFICANT ADVERSE ENVIRONMENTAL IMPACTS THAT CAN BE FEASIBLY MITIGATED OR AVOIDED, for which the decision-maker must make "findings" under Section 15091 of the State CEQA Guidelines (as amended) if the project is approved.

Impact Summary	Middle Fork Sand and Gravel Deposits	Old Cool Quarry	Cool Quarry Amphibolite	Bear River/ Chevreaux	Mississippi Bar	Yuba River
LAND USE	**Impact:** No Class II land use impacts have been identified.	**Impact:** Expanded operation could conflict with surrounding residential uses.	**Impact:** Same as the Old Cool Quarry.	**Impact:** No Class II land use impacts have been identified.	**Impact:** No Class II land use impacts have been identified.	**Impact:** Expanded hours of operation could result in noise impacting residents in the area.
PUBLIC HEALTH AND SAFETY	**Impact:** Potential safety problems with persons climbing or riding conveyor or mining equipment.	**Impact:** Same as described for the Middle Fork Sand and Gravel Deposit alternative.	**Impact:** Same as described for the Middle Fork Sand and Gravel Deposit alternative.	**Impact:** No Class II impacts to public health and safety have been identified (see Class I).	**Impact:** No Class II impacts to public health and safety have been identified (see Class I).	**Impact:** No Class II impacts to public health and safety have been identified (see Class I).
	Impact: Short-term impact caused by increased truck traffic in the Middle Fork vicinity.		**Impact:** Unauthorized persons gaining access to excavation and stockpile areas.			
	Impact: Potential flooding of mining operation sites.					
	Impact: Operational risks such as fuel storage, equipment malfunctions, and public access are common to all sites.	**Impact:** Same as for Middle Fork Sand and Gravel alternative.	**Impact:** Same as for Middle Fork Sand and Gravel alternative.	**Impact:** Same as for Middle Fork Sand and Gravel alternative.	**Impact:** Same as for Middle Fork Sand and Gravel alternative.	**Impact:** Same as for Middle Fork Sand and Gravel alternative.

TABLE 2-1. ALTERNATIVE AGGREGATE SOURCES IMPACT SUMMARY TABLE

CLASS II. (CONTINUED).

Impact Summary	Middle Fork Sand and Gravel Deposits	Old Cool Quarry	Cool Quarry Amphibolite	Bear River/ Chevreaux	Mississippi Bar	Yuba River
WATER QUALITY	<u>Impact</u>: Impacts associated with mining the Middle Fork would include elevated levels of turbidity and increased contaminant levels.	<u>Impact</u>: Potential water quality impacts are similar to other quarry operations such as sediment discharges into receiving waters; however, existing processing operation and settling pits are located away from any waterways.	<u>Impact</u>: Potential significant water quality impacts could occur during rainy season depending on the location of stock-piled overburden.	<u>Impact</u>: Water quality degradation associated with increased excavation and processing.	<u>Impact</u>: Increased erosion, caused by surface runoff, excavation, and processing, could violate existing environmental commitments.	<u>Impact</u>: Water quality degradation associated with increased excavation and processing.
AIR QUALITY	<u>Impact</u>: No Class II impacts to air quality have been identified (see Class I).	<u>Impact</u>: No Class II impacts to air quality have been identified (see Class I).	<u>Impact</u>: No Class II impacts to air quality have been identified (see Class I).	<u>Impact</u>: No Class II impacts to air quality have been identified (see Class I).	<u>Impact</u>: No Class II impacts to air quality have been identified (see Class I).	<u>Impact</u>: No Class II impacts to air quality have been identified (see Class I).
FISH, VEGETATION, AND WILDLIFE	<u>Impact</u>: Project implementation would potentially impact the Valley Elderberry Longhorn Beetle.	<u>Impact</u>: No Class II impacts to fish, vegetation and wildlife have been identified.	<u>Impact</u>: The development of new quarry site would result in loss of vegetation.	<u>Impact</u>: Assumption that new areas would be quarried resulted in an estimated loss of 245 acres of Oak Woodlands along with the cumulative loss of approximately 189 habitat units.	<u>Impact</u>: The loss of approximately 30 acres of riparian woodland along with cumulative impacts to wildlife associated with the loss of cover type would result in the loss of 24 habitat units.	<u>Impact</u>: Under the assumption that 155 acres would be disturbed and that vegetation covers approximately 10% of the site, a total of 15.5 acres of riparian habitat would be lost.

TABLE 2-1. ALTERNATIVE AGGREGATE SOURCES IMPACT SUMMARY TABLE

CLASS II. (CONTINUED).

Impact Summary	Middle Fork Sand and Gravel Deposits	Old Cool Quarry	Cool Quarry Amphibolite	Bear River/ Chevreaux	Mississippi Bar	Yuba River
TRANSPORTATION	<u>Impact</u>: Short-term impacts related to increases in daily trips on SR 49, SR 193, and Auburn City streets, thereby lowering the levels of service in these areas.	<u>Impact</u>: Transportation impacts are the same as Middle Fork Sand and Gravel alternative.	<u>Impact</u>: Transportation impacts are the same as Middle Fork Sand and Gravel alternative.	<u>Impact</u>: No Class II transportation impacts have been identified (see Class I).	<u>Impact</u>: No Class II transportation impacts have been identified (see Class I).	<u>Impact</u>: No Class II transportation impacts have been identified (see Class I).
NOISE	<u>Impact</u>: No Class II noise impacts have been identified (see Class I).	<u>Impact</u>: Blasting.	<u>Impact</u>: Blasting.	<u>Impact</u>: Blasting.	<u>Impact</u>: Same as described for Middle Fork Sand and Gravel alternative.	<u>Impact</u>: Same as described for Middle Fork Sand and Gravel alternative.
RECREATION	<u>Impact</u>: Cumulative loss of the Middle Fork bars would impact hiking, riding, and special endurance events.	<u>Impact</u>: No Class II impacts to recreation have been identified.	<u>Impact</u>: Impacts associated with this alternative are the same as the Old Cool Quarry.	<u>Impact</u>: No significant impacts to recreation have been identified for this alternative.	<u>Impact</u>: Increased truck traffic would impact equestrian trails.	<u>Impact</u>: No significant impacts to recreation have been identified for this alternative.
VISUAL RESOURCES	<u>Impact</u>: No Class II impacts to visual resources have been identified.	<u>Impact</u>: Increased operations of this existing facility would increase the amount of fugitive dust.	<u>Impact</u>: No Class II impacts to visual resources have been identified.	<u>Impact</u>: No Class II impacts to visual resources have been identified.	<u>Impact</u>: Possible increases in fugitive dust and noise may occur.	<u>Impact</u>: No significant visual impacts would occur with implementation of this alternative.
IN-STREAM IMPACTS (only for Middle Fork Sand and Gravel Deposits)	No Class II in-stream impacts have been identified (see Class I).					

3.0 PROJECT DESCRIPTION

3.1 AGGREGATE SOURCE LOCATIONS AND SITE DESCRIPTIONS

Figure 1-1 shows the regional locations of the various potential aggregate sources selected for further review. Cursory site descriptions are provided below.

3.1.1 MIDDLE FORK SAND AND GRAVEL DEPOSITS

The aggregate deposits are contained within a series of gravel bars located along a seven-mile section of the Middle Fork, starting approximately one mile upstream from the confluence of the Middle and North Forks (Figure 3-1, Site Map of the Middle Fork Bars). Table 3-1 summarizes the characteristics of the Middle Fork bars. The deposit is a series of lateral and point bars together containing an estimated 8.6 million cubic yards of sand and gravel subequally distributed between the 10 bars. Mammoth Bar, the furthest downstream of the bars, is approximately five river miles from the proposed dam site. Cherokee Bar, the uppermost bar, is approximately twelve river miles from the dam site. The bars are located within unincorporated portions of El Dorado and Placer Counties.

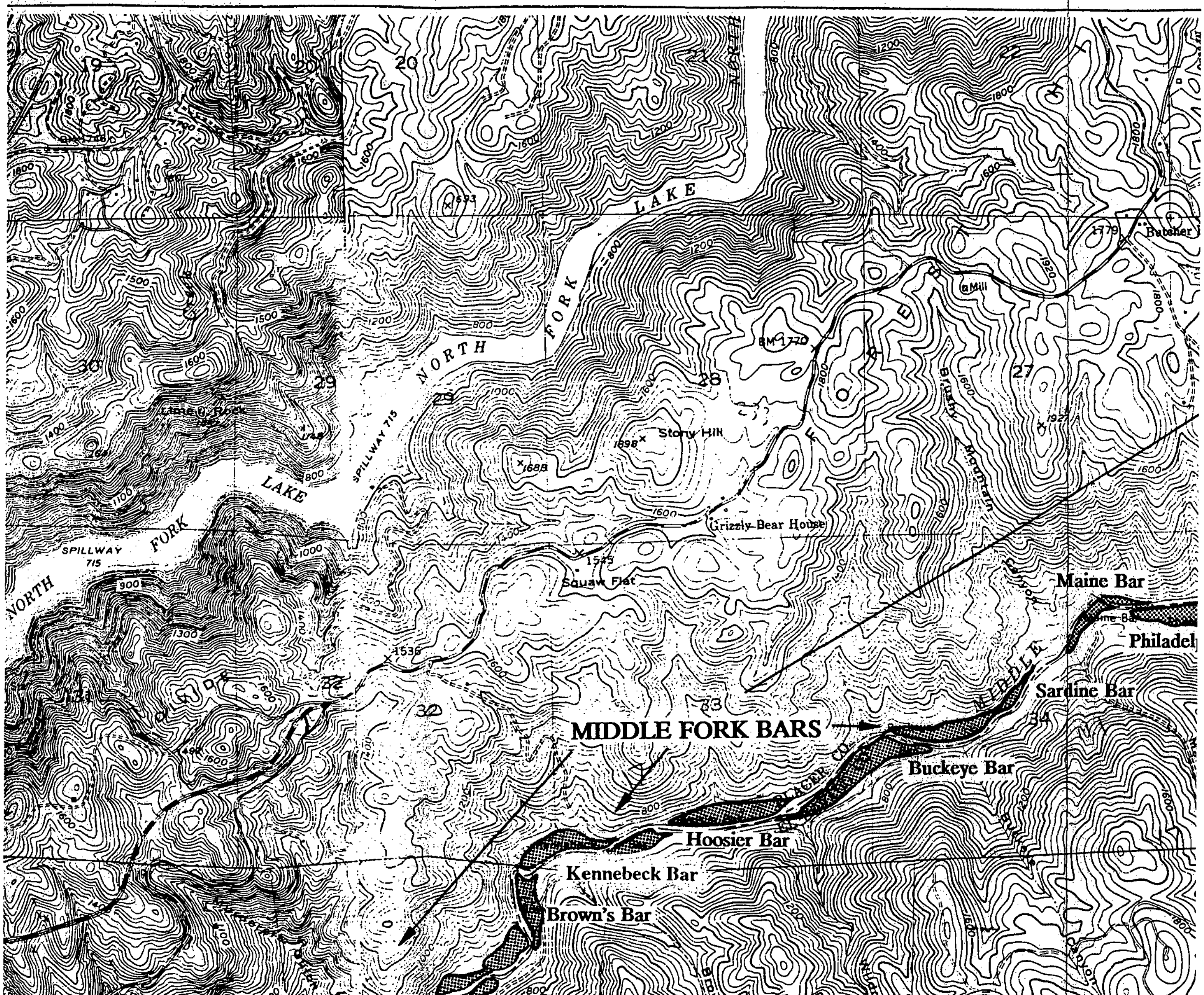
The following descriptions of the bar deposits are based on data gathered during preliminary engineering studies conducted for the original multi-purpose project (USBR, 1968; 1976).

Mammoth Bar. This is the first major bar located upstream of the Auburn dam site (Figure 3-1). Figures 3-2 and 3-3 are aerial views of Mammoth Bar. The bar covers an area of approximately 971,250 square feet. Studies indicate that four percent of the deposit consists of materials larger than six inches with a maximum particle size of 12 inches. The fines (minus #200 sieve) content of the pitrun material averages 9.5 percent. The pitrun gradation classifies as a GP-GM (poorly graded gravel with silt). The sand content of the bar is estimated at 36 percent. The average depth of the bar was estimated at 31.4 feet (depth to bedrock in clamshell test holes). The projected yield of the bar is 1,129,500 cubic yards of aggregate.

Texas Bar. Texas Bar is located upstream of Mammoth Bar and lies along both banks of the river (Figure 3-1). Figure 3-4 is an aerial view of the bar. The average gradation (pitrun composition) is nearly the same as that of Mammoth Bar. Texas Bar covers an area of approximately 996,000 square feet and has an average depth to bedrock of 31.2 feet. The projected yield of the bar, at 1,150,900 cubic yards of aggregate, is the second largest yield of the ten bars.

Browns Bar. Browns Bar is located along the south bank of the Middle Fork immediately upstream of Texas Bar (Figure 3-1). Figure 3-5 is an aerial view of the bar. The pitrun material classifies as a GW-GM (well-graded gravel with silt), and contains approximately 36 percent sand and 7 percent fines. The bar covers an area of approximately 599,000

Source: USGS Topographical Map, Greenwood and Auburn Quadrangles



Middle Fork Bars Location Map

FUGRO McClelland

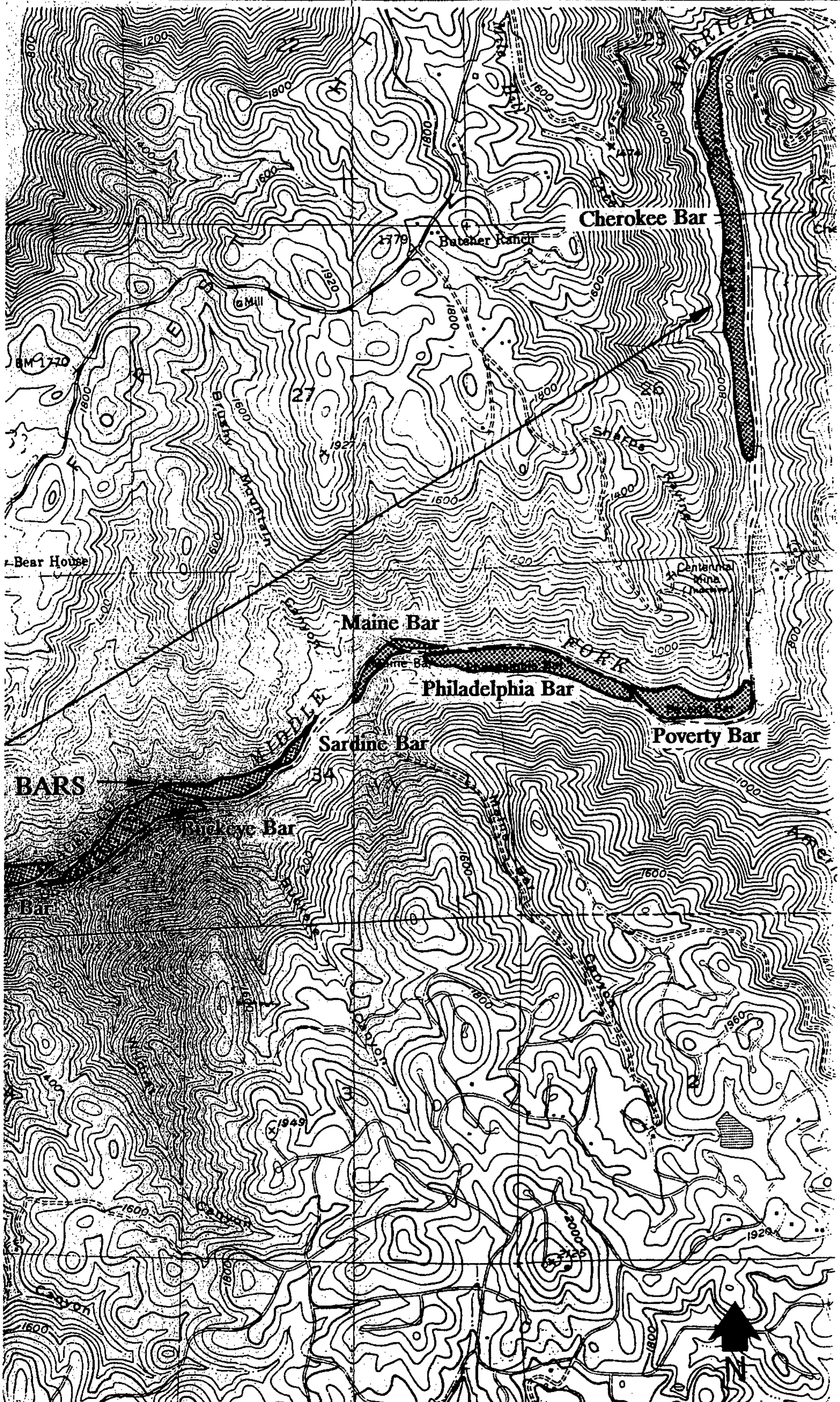


FIGURE 3-1

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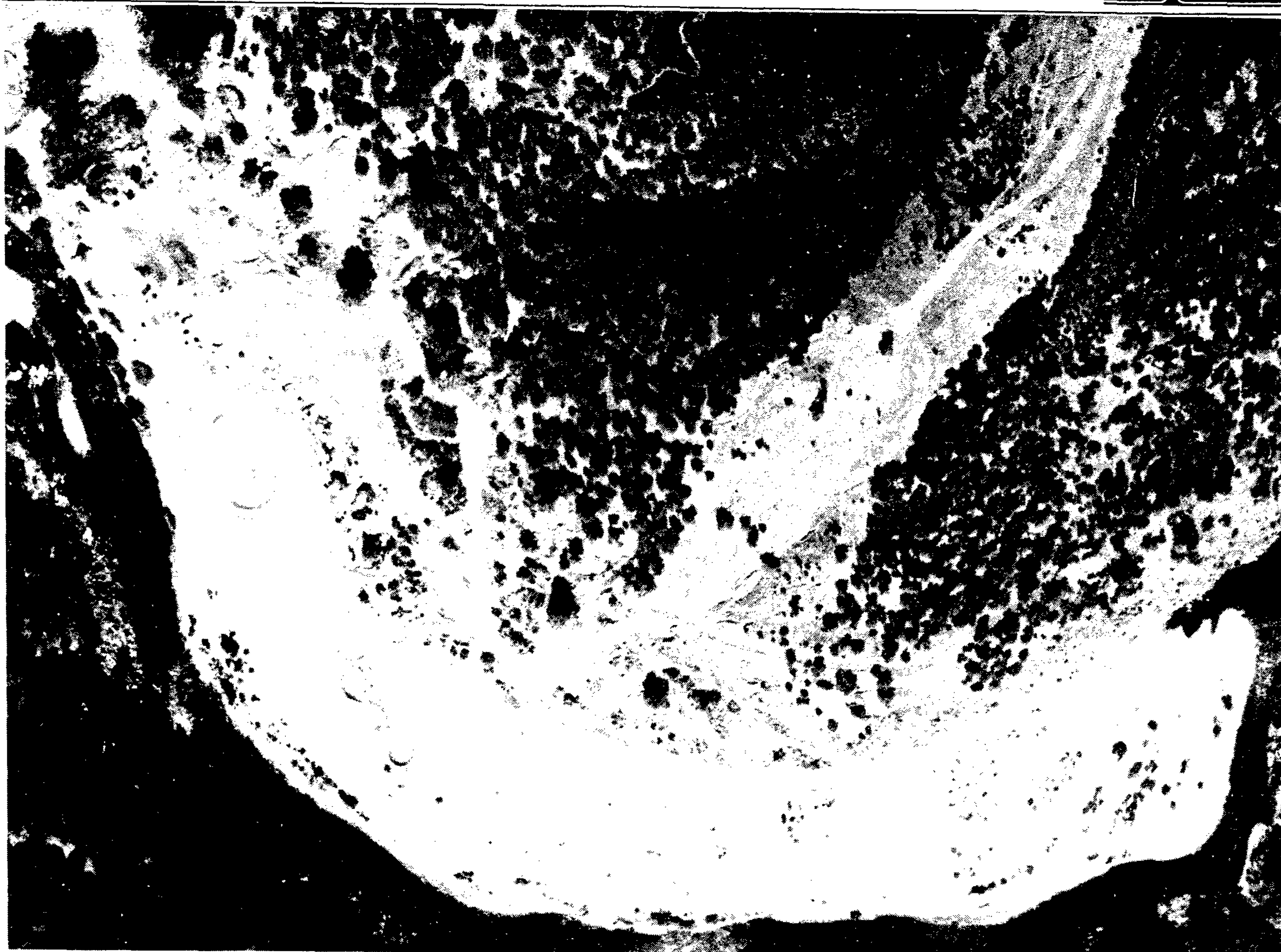
TABLE 3-1
SUMMARY TABLE OF MIDDLE FORK AMERICAN RIVER SAND AND GRAVEL DEPOSITS

BAR NAME	SIZE (SQ.FT)	DEPTH* (FT.)	PROJECTED YIELD (CU.FT.)
Mammoth	971,250	31.4	1,129,500
Texas	996,000	31.2	1,150,900
Browns	599,000	30.4	674,400
Kennebeck	719,000	31.5	839,000
Hoosier	649,000	25.9	622,500
Buckeye	1,104,000	27.6	1,128,000
Maine	249,000	21.0	194,000
Philadelphia	907,000	23.7	705,000
Poverty	1,152,000	27.5	1,173,000
Cherokee	1,484,000	18.0	989,000
Exposed Bar Total	8,830,250		8,605,300
Average Depth		26.82	

* Depth to bedrock

FUGRO McClelland



Aerial Photo - Mammoth Bar

Source: U S Bureau of Reclamation

FIGURE 3-2

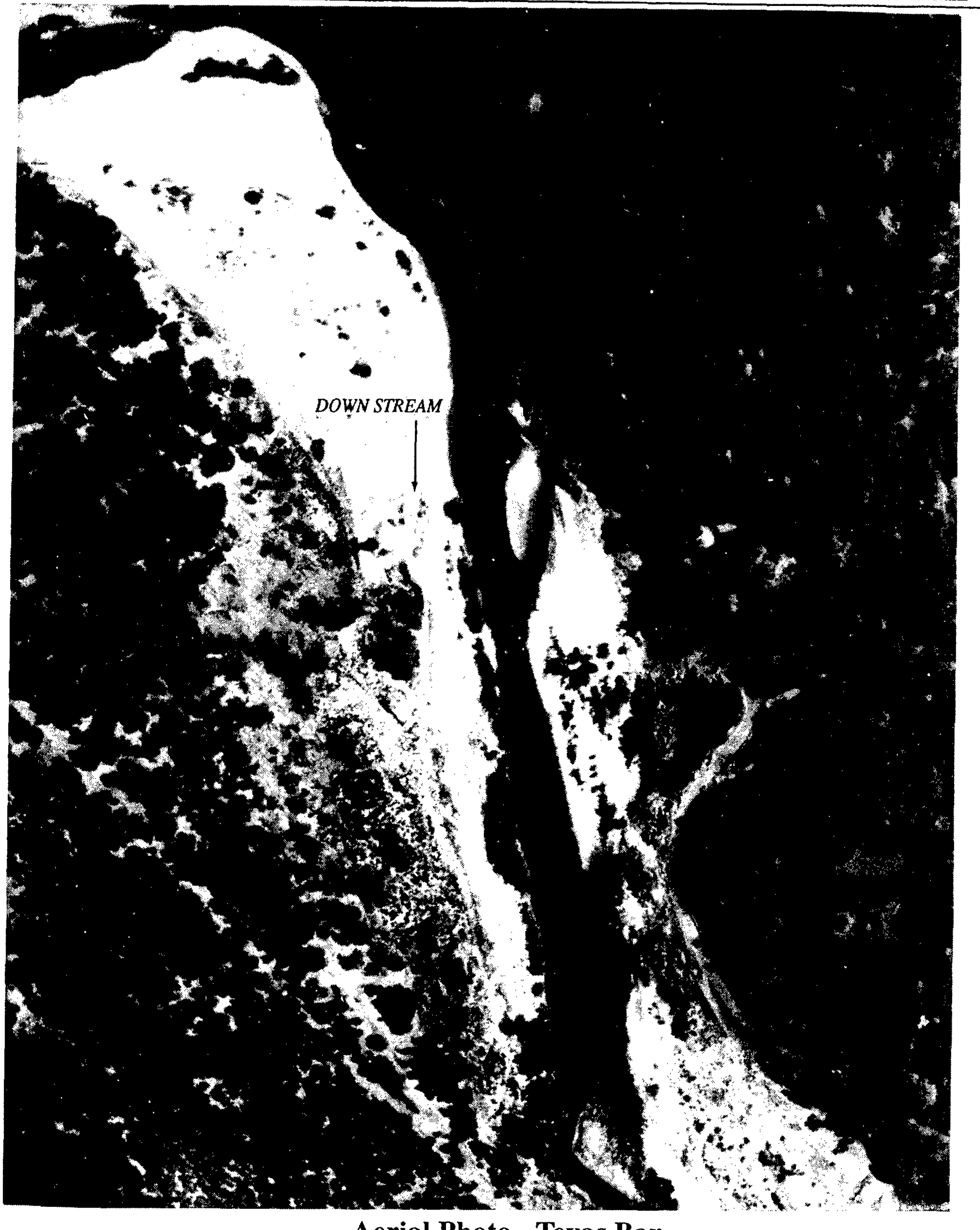
FUGRO McClelland

Oblique Aerial Photo - Mammoth Bar in Foreground, Texas and Brown's Bars in Background

Source: U S Bureau of Reclamation

FIGURE 3-3

FUGRO McClelland



Aerial Photo - Texas Bar

Source: U S Bureau of Reclamation

FIGURE 3-4

FUGRO McClelland

Aerial Photo - Brown's Bar

Source: U S Bureau of Reclamation

FIGURE 3-5

square feet with an average depth to bedrock of 30.4 feet. The bar contains approximately 674,400 cubic yards of aggregate.

Kennebeck Bar. Most of the material from Kennebeck Bar is located along the south bank of the Middle Fork, just upstream of Browns Bar (Figure 3-1). Figure 3-6 is an aerial view of the bar. The pitrun material classifies as a SP-SM (poorly graded sand with silt), and contains approximately 51 percent sand and 6 percent fines. The bar covers an area of approximately 719,000 square feet with an average depth to bedrock of 31.5 feet. It is estimated that the bar contains approximately 839,000 cubic yards of material.

Hoosier Bar. Hoosier Bar is located along the north bank of the Middle Fork (Figure 3-1). Figure 3-7 is an aerial view of the bar. The pitrun material classifies as a GP-GM (poorly graded gravel with silt), and contains 40 percent sand and 6 percent fines. The bar covers an area of approximately 649,000 square feet with an average depth to bedrock of 25.9 feet. It is estimated that the bar contains approximately 622,500 cubic yards of material.

Buckeye Bar. The materials of Buckeye Bar are evenly distributed along both banks of the Middle Fork (Figure 3-1). Figure 3-8 is an aerial view of the bar. The pitrun material classifies as a GW-GM (well-graded gravel with silt), and contains approximately 32 percent sand and 7 percent fines. The bar covers an area of approximately 1,104,000 square feet with an average depth to bedrock of 27.6 feet. It is estimated that the Buckeye Bar contains 1,128,000 cubic yards of aggregate.

Sardine Bar. Sardine Bar lies within the channel of the Middle Fork (Figure 3-1) and is rather small in size when compared to the other bars. No exploration borings have been conducted specifically within Sardine Bar. The volume of aggregate material available within the bar is therefore undetermined and is included with the estimated quantity of material (1 million cubic yards) which lie within the river channel itself.

Maine Bar. Maine Bar, with an estimated area of 249,000 square feet, is the smallest of the ten exposed bars. It lies along the left bank of the Middle Fork just upstream of Sardine Bar (Figure 3-1). Figure 3-9 is an aerial view of the bar. The material classifies as GP-GM (poorly graded gravel with silt) and contains approximately 15 percent sand and 7 percent fines. The bar has an average depth to bedrock of 21.0 feet and contains approximately 194,000 cubic yards of material.

Philadelphia Bar. This bar is located along the north bank of the Middle Fork, just upstream of Maine Bar (Figure 3-1). Figures 3-10 and 3-11 show aerial views of the bar. The material classifies as GW-GM (well graded gravel with silt) and contains 28 percent sand and 10 percent fines. Given the average depth to bedrock of 23.7 feet and an area of 907,000 square feet, the bar contains about 705,000 cubic yards of material.

← DOWN STREAM

Aerial Photo - Kennebeck Bar

Source: U S Bureau of Reclamation

FIGURE 3-6

← DOWN STREAM

Aerial Photo - Hoosier Bar

Source: U S Bureau of Reclamation

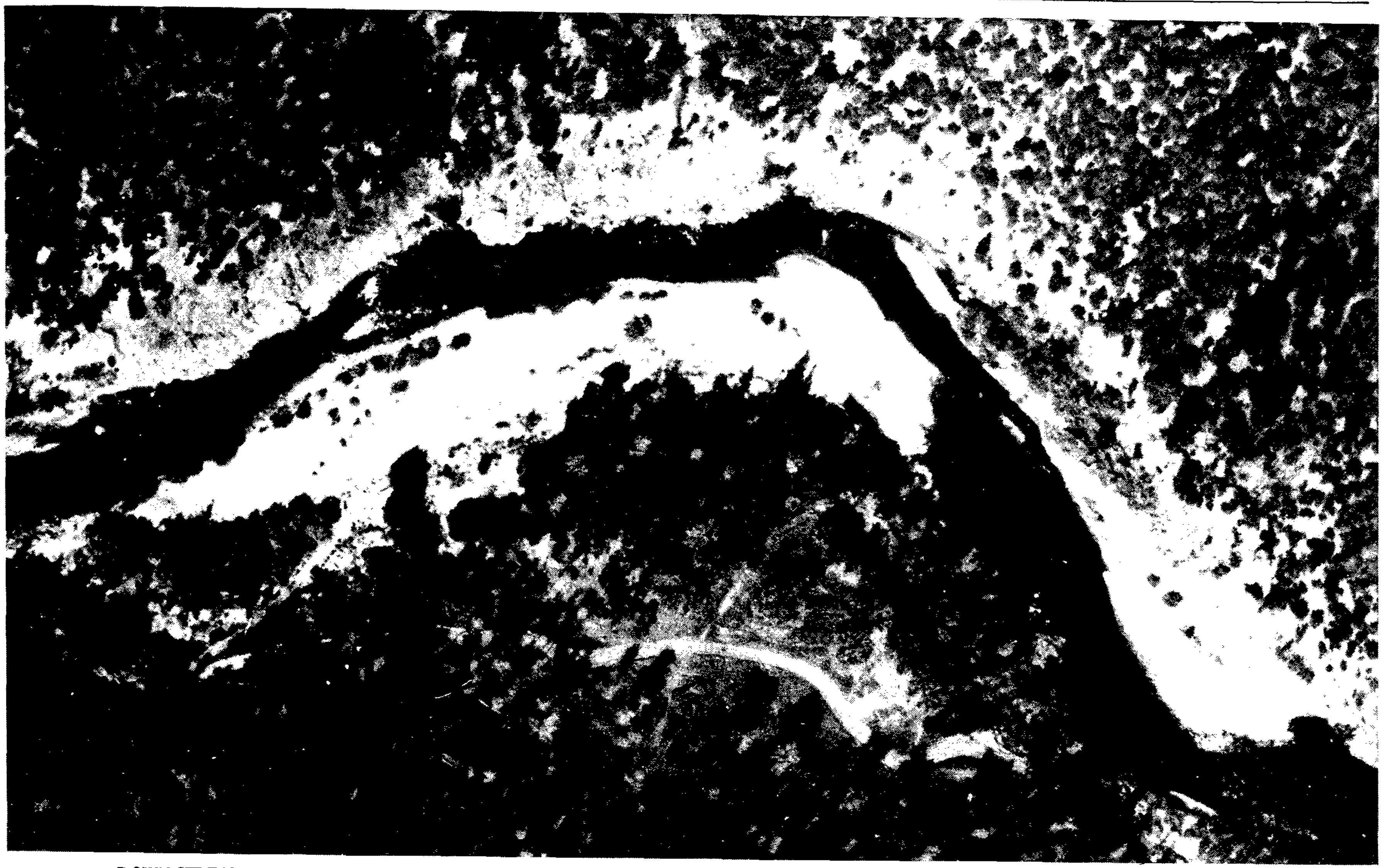
FIGURE 3-7

← DOWN STREAM

Aerial Photo - Buckeye Bar

Source: U S Bureau of Reclamation

FIGURE 3-8



← DOWN STREAM

Aerial Photo - Maine Bar

Source: U S Bureau of Reclamation

FIGURE 3-9

← DOWN STREAM

Aerial Photo - Philadelphia Bar

Source: U S Bureau of Reclamation

FIGURE 3-10

View Looking Upstream - Philadelphia Bar in Foreground, Poverty Bar in Background

Source: U S Bureau of Reclamation

FIGURE 3-11

Poverty Bar. The materials of Poverty Bar are evenly distributed along both banks of the Middle Fork. The bar is located on a bend of the river just upstream of Philadelphia Bar (Figure 3-1). Figures 3-11 and 3-12 show aerial views of the bar. The material classifies as SP-SM (poorly graded sand with silt) and contains 52 percent sand and 5 percent fines. The average depth to bedrock is 27.5 feet. The bar, with an area of 1,152,000 square feet, is the second largest and the projected yield of 1,173,000 cubic yards of aggregate is the highest yield.

Cherokee Bar. Cherokee Bar is located upstream of Poverty Bar. The majority of the deposit lies along the north bank of the Middle Fork (Figure 3-1). Figure 3-13 is an aerial view of the bar. This is the longest of the identified bars. The material classifies as a GC (clayey gravel with sand) and contains 22 percent sand and 12 percent fines. Based on an area of approximately 1,484,000 square feet with an average depth to bedrock of 18.0 feet, the bar is estimated to contain 989,000 cubic yards of material.

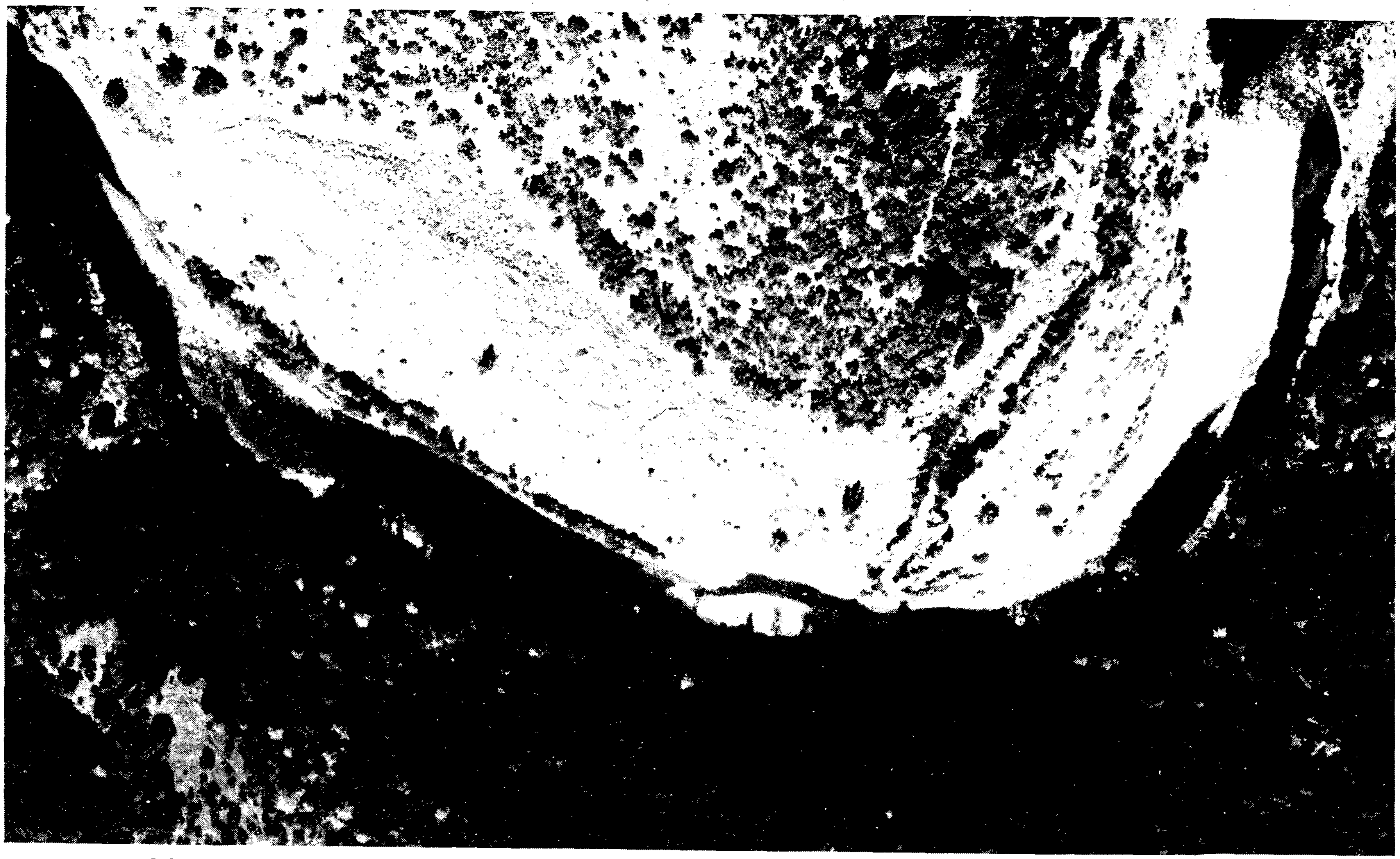
River Channel. The river channel of the Middle Fork American River is estimated to contain at least 968,000 cubic yards of material. This assumes an average thickness of six feet for the channel deposits and a channel area of 4,357,000 square feet (this does not include the bar areas).

3.1.2 OLD COOL QUARRY (SPRECKLES)

Originally developed to supply lime to the sugar industry, the Old Cool Quarry is located 1.5 miles north of the town of Cool on the east side of Highway 49 in El Dorado County (Figure 3-14). Figure 3-15 is an aerial view of the quarry. It is in T12N, R9E, NE 1/4 Sec. 7 MDBM (Auburn 7.5 minute quadrangle). The site is in a disturbed condition owing to the long history of quarrying activities. The upper (southern) part of the property is nearly flat. Processing and storage of the various products occurs in this area. The western and northern portions of the property have large, steeply-walled pits as a result of the mining activities. Overburden removed from areas targeted for quarrying is used to fill the old pits.

3.1.3 COOL QUARRY AMPHIBOLITE

Located immediately west of the existing Cool Quarry (Figure 3-14), this area is located in the SE 1/4 of the SW 1/4, Section 6 and also the SW 1/4 of the SE 1/4, Sec. 6, T. 12 N., R. 9 E. (Auburn 7.5 minute quadrangle). Figures 3-15 and 3-16 show aerial views of the possible quarry site. Topographically, the site occupies the nose of a plunging ridge which terminates at the canyon bottom. The terrain is steep and the vegetation heavy. Bedrock is exposed throughout the site and in road cuts crossing the site. The fire road crossing the site has sustained severe erosion damage as a result of ineffective culverting of stormwater runoff.



← DOWN STREAM

Aerial Photo - Poverty Bar

Source: U S Bureau of Reclamation

FIGURE 3-12

Aerial Phot

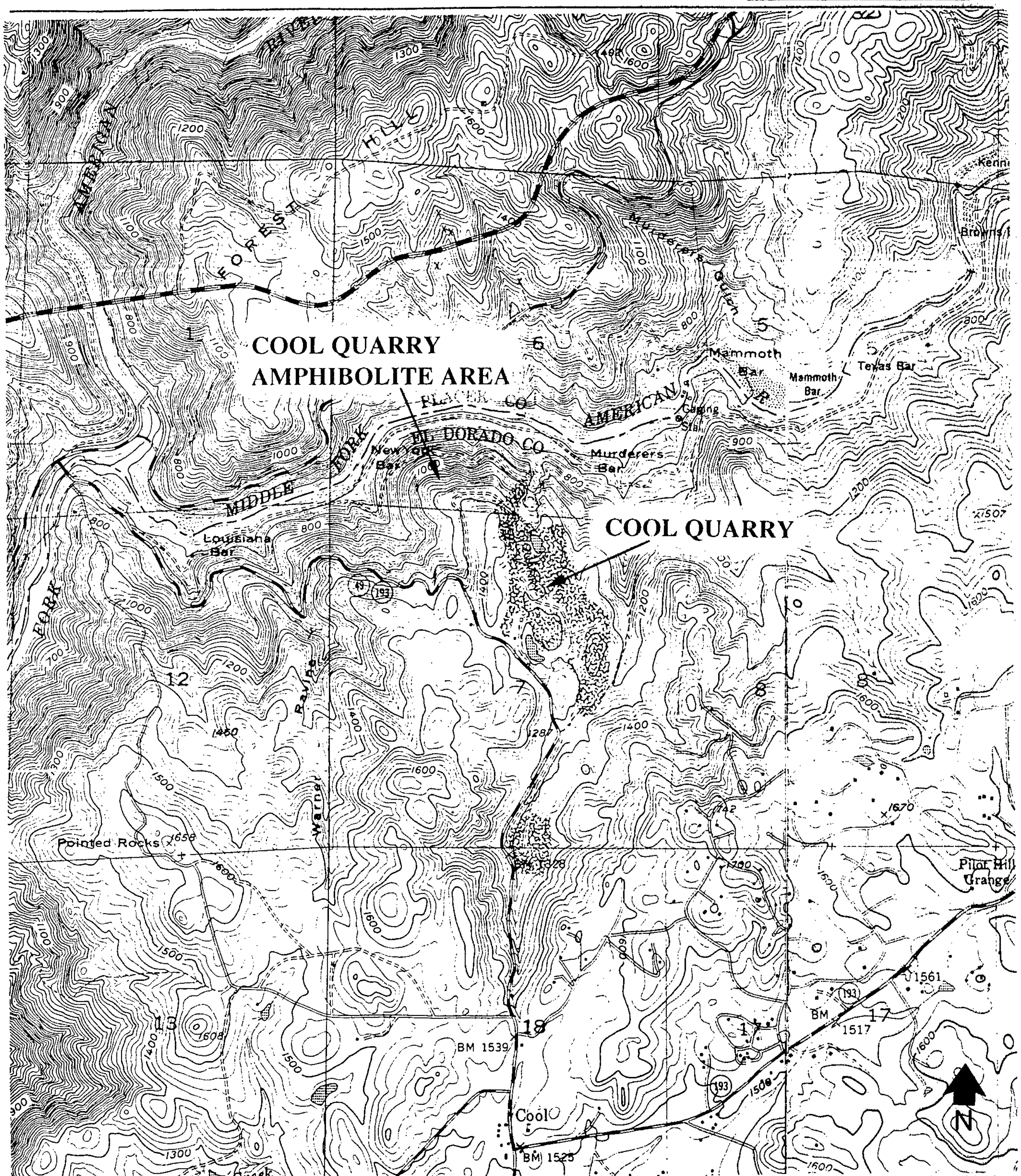
Source: U S Bureau of Reclamation

1

Aerial Photo - Cherokee Bar

DOWN STREAM ⟶

Fugro McClelland

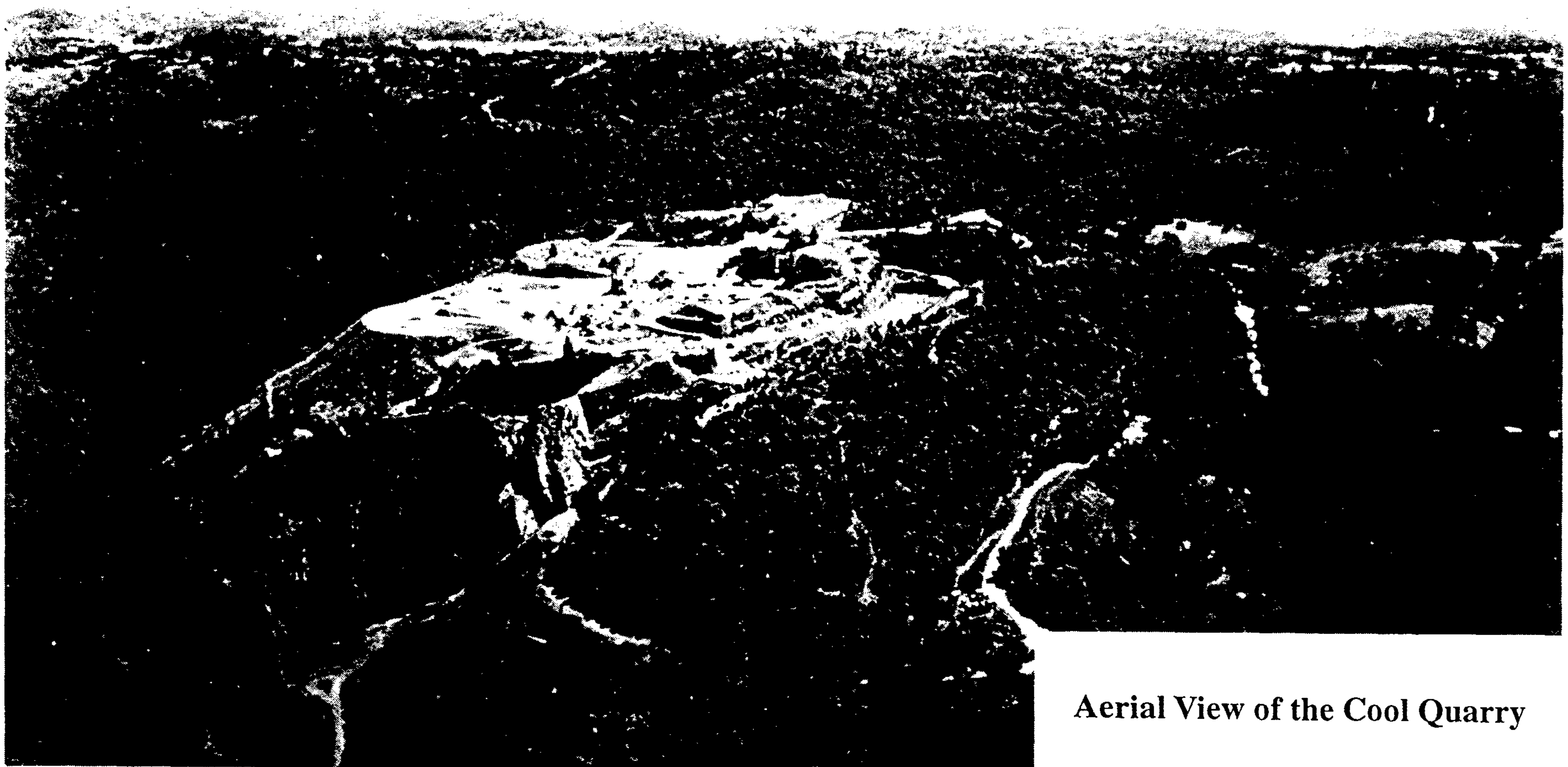


Scale: 1:24,000

Cool Quarry & Cool Quarry Amphibolite Location Map

Source: USGS Topographical Map, Auburn Quadrangle

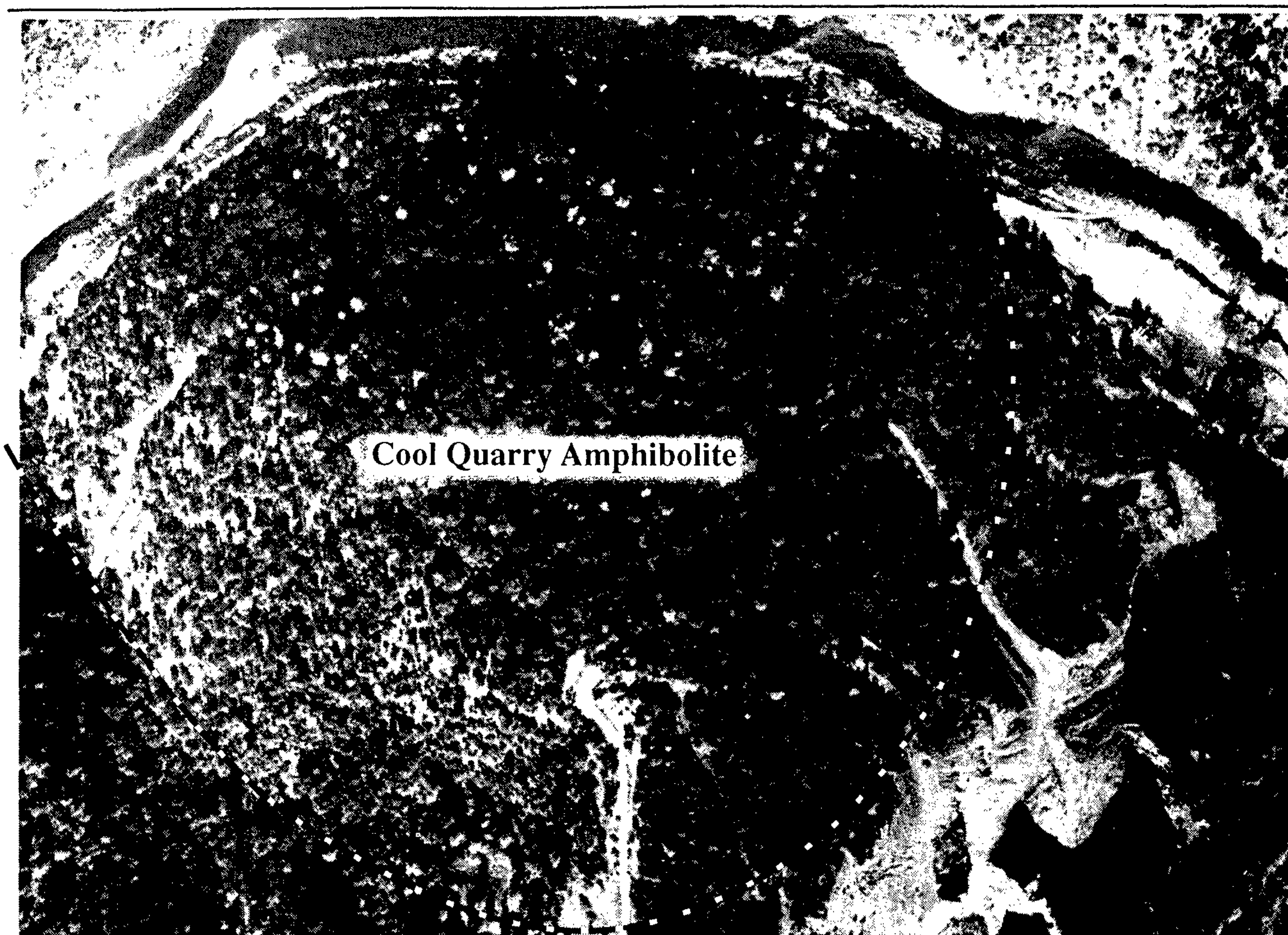
FIGURE 3-14



Aerial View of the Cool Quarry

Source: U S Army Corps of Engineers

FIGURE 3-15



Aerial Photo - Cool Quarry Amphibolite Area

Source: U S Bureau of Reclamation

FIGURE 3-16

3.1.4 BEAR RIVER AND CHEVREAUX QUARRY

Lake Combie-Bear River Alluvial Deposits - Holocene sediments currently being mined from Lake Combie and the Bear River are predominantly derived from mining debris produced when gold-bearing gravel was hydraulically mined in the Sierra Nevada during the late 1800s. Much of this mining debris has since washed downstream and has concentrated behind Lake Combie Dam, which was built in 1928. The fluvial transport of this older mining debris has naturally graded, rounded, and washed away the softer materials leaving a deposit consisting of hard, durable, well-rounded, mature quartzose sands and gravels in the study area. Replenishment of the quartzose mining debris has continued but it is expected to gradually decrease in the future, in part due to Rollins Reservoir Dam which was built upstream in the mid-1960s.

Current alluvial mining operations of the Bear River-Lake Combie deposit extend upstream from Lake Combie Dam over two miles. Average width of the deposit is about 800 feet. This deposit extends along and is divided roughly in half by the boundary line between Placer and Nevada Counties.

The Chevreaux Quarry is located about 2 miles north of the town of Meadow Vista, approximately 12 miles north of Auburn in Placer County. Figure 3-17 shows the Chevreaux operations and the local limits of the designated Mineral Resource Zone. It is in T41N, R9E, SW 1/4 Sec 30, MDBM (Lake Combie 7.5 minute quadrangle). Sand is obtained from dredging in Lake Combie and crushed rock from a quarry near the Bear River. The natural state of the site is disturbed due to historic mining activity.

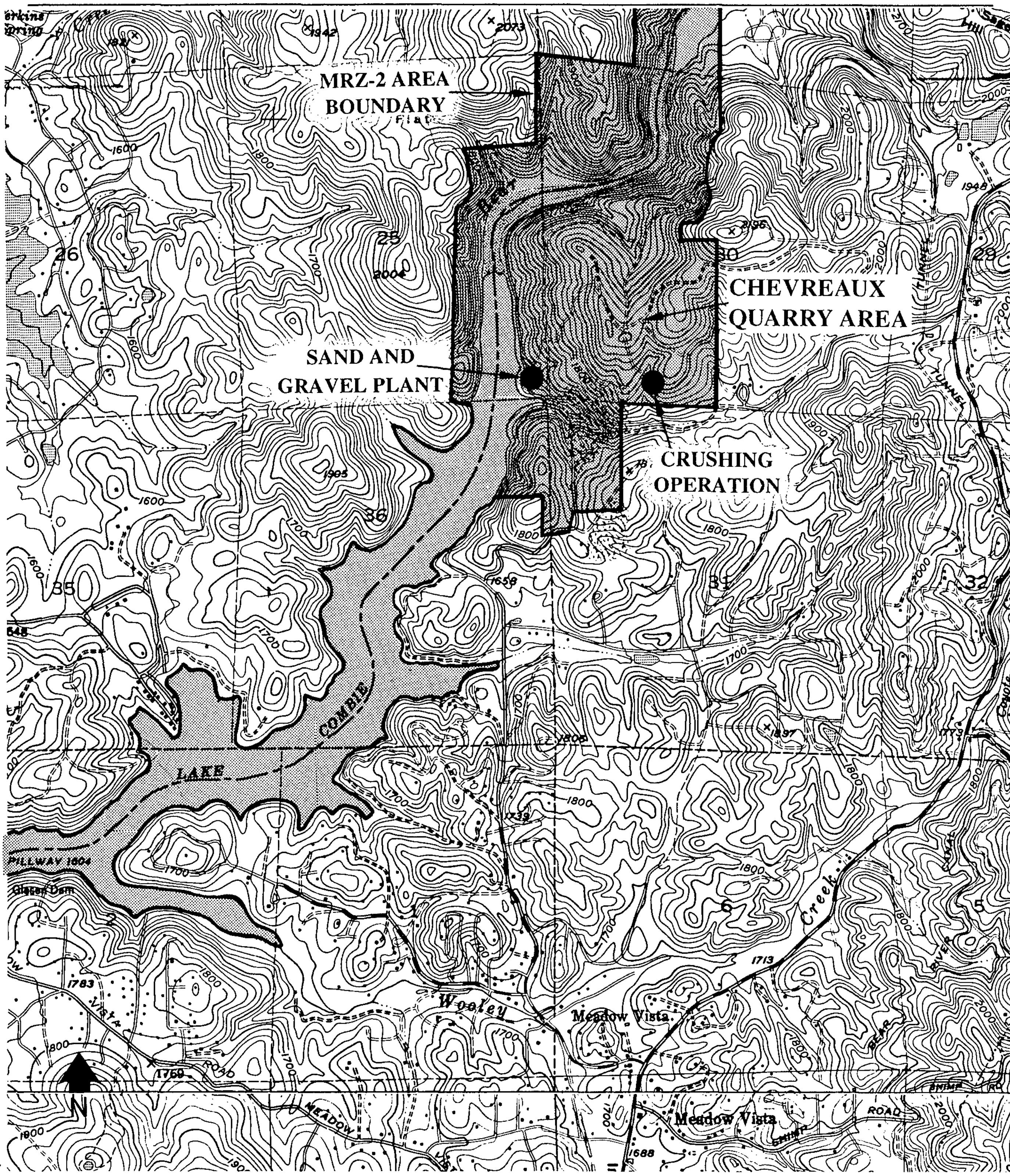
3.1.5 MISSISSIPPI BAR

Mississippi Bar is located about 1 mile upstream from Nimbus Dam on the north side of Lake Natoma in the American River Parkway. The current sand and gravel mining operation is situated on 160 acres of federally-owned land administered by the USBR. Figure 3-18 shows the area currently designated for aggregate extraction. It is bounded on the north, west and south by State of California lands, and by Federal lands on the east (see Section 4.1, Land Use). The land surface consists of extensive dredge tailings intermixed with clay deposits ("slickens") on which extensive vegetation has grown.

Teichert Aggregates maintains and operates a rock crushing plant and an asphalt batch plant on state lands to the west under an agreement with the California Department of Parks and Recreation. The agreement provides for harvesting the dredge tailings as rock aggregate, while also rendering the land as an undulating surface characterized by excavated lagoons or ponds.

Access to the currently permitted site is via Main Avenue where east Sunset Avenue ends in Orangevale. Land uses in the surrounding vicinity include medium- to low-density

Fugro McClelland

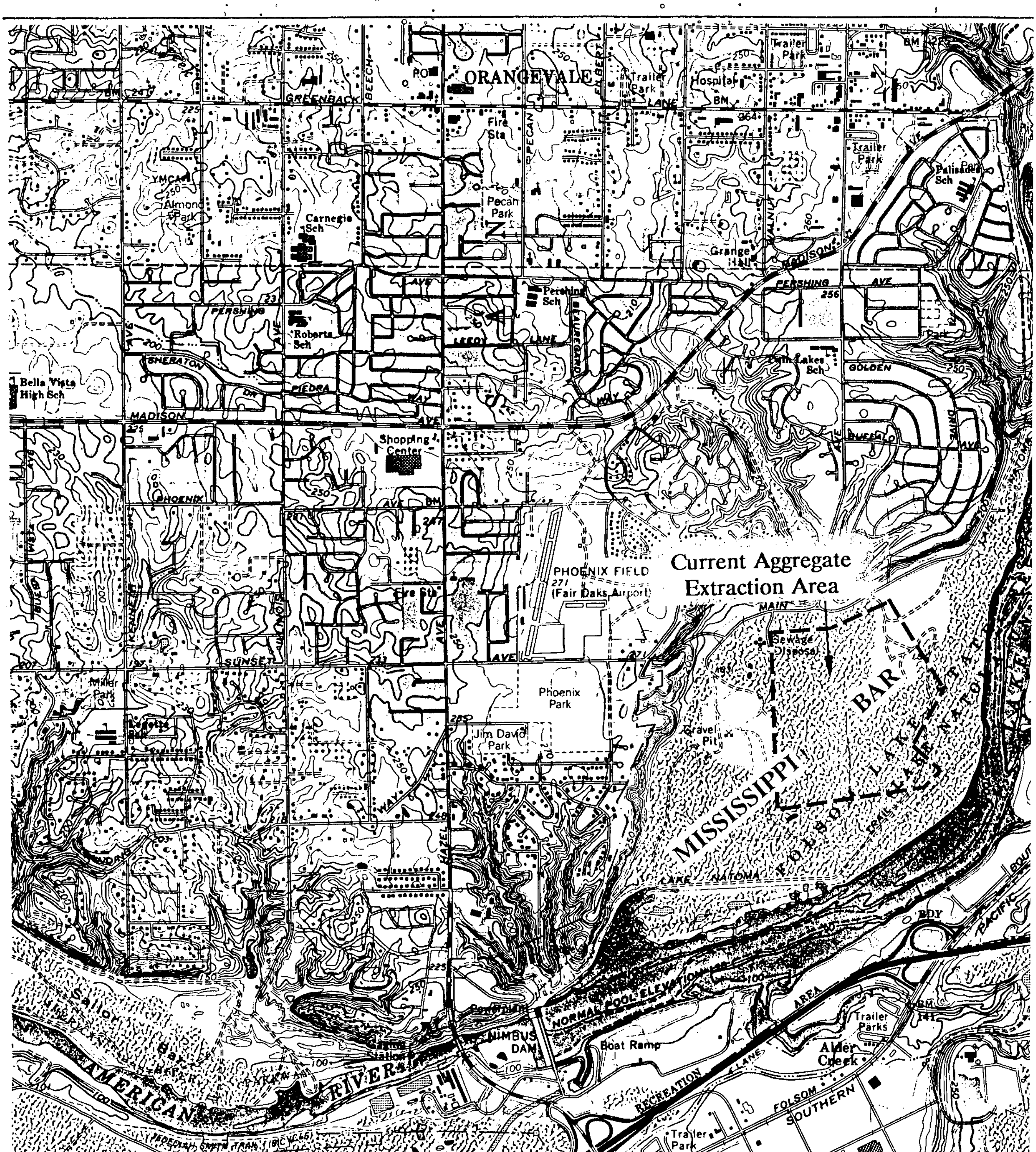


Scale: 1: 24,000

Chevreaux Quarry Location Map

Source: USGS Topographical Map and Dupras, 1983

FIGURE 3-17

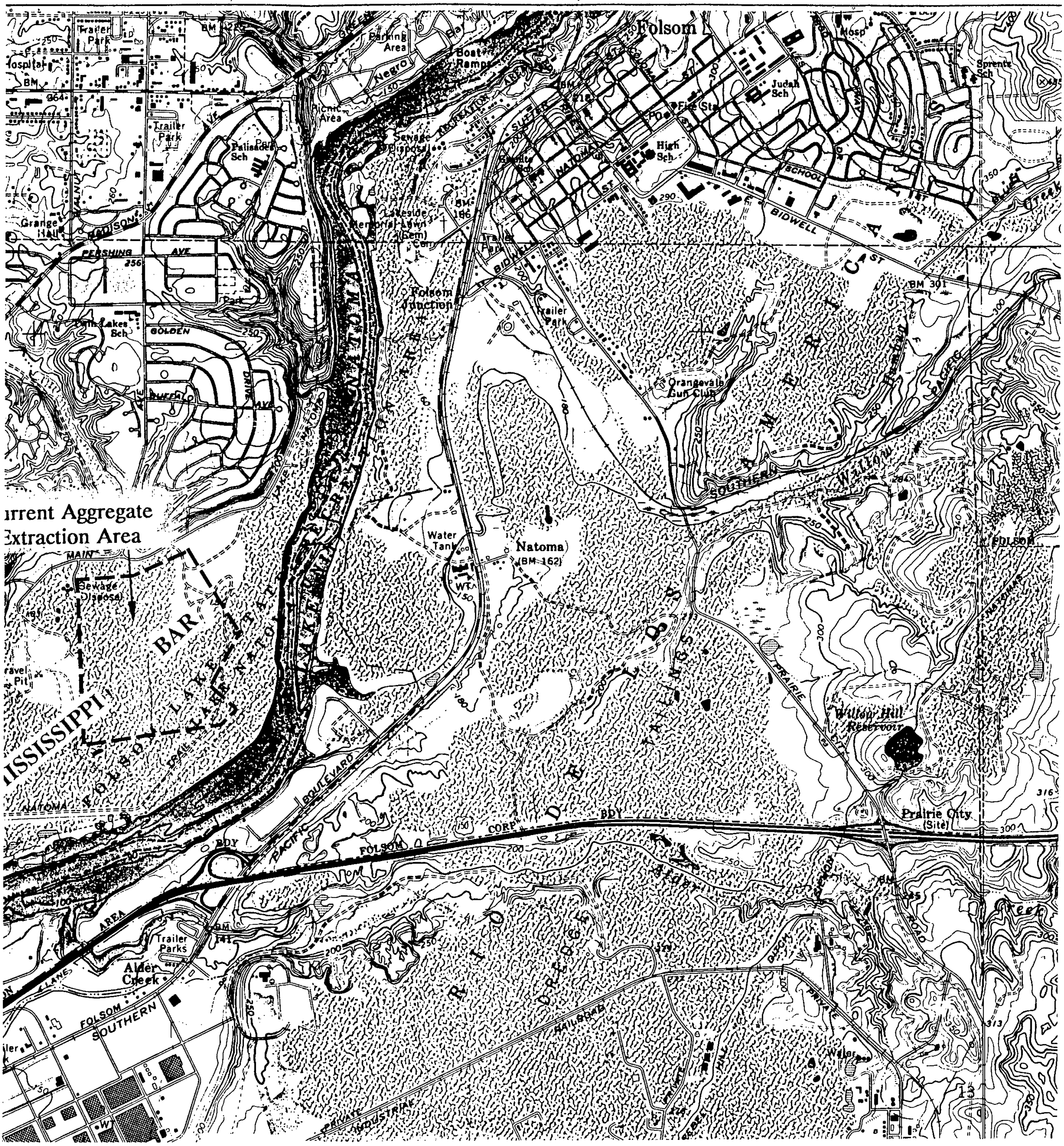


Approx. Scale

1 Mile

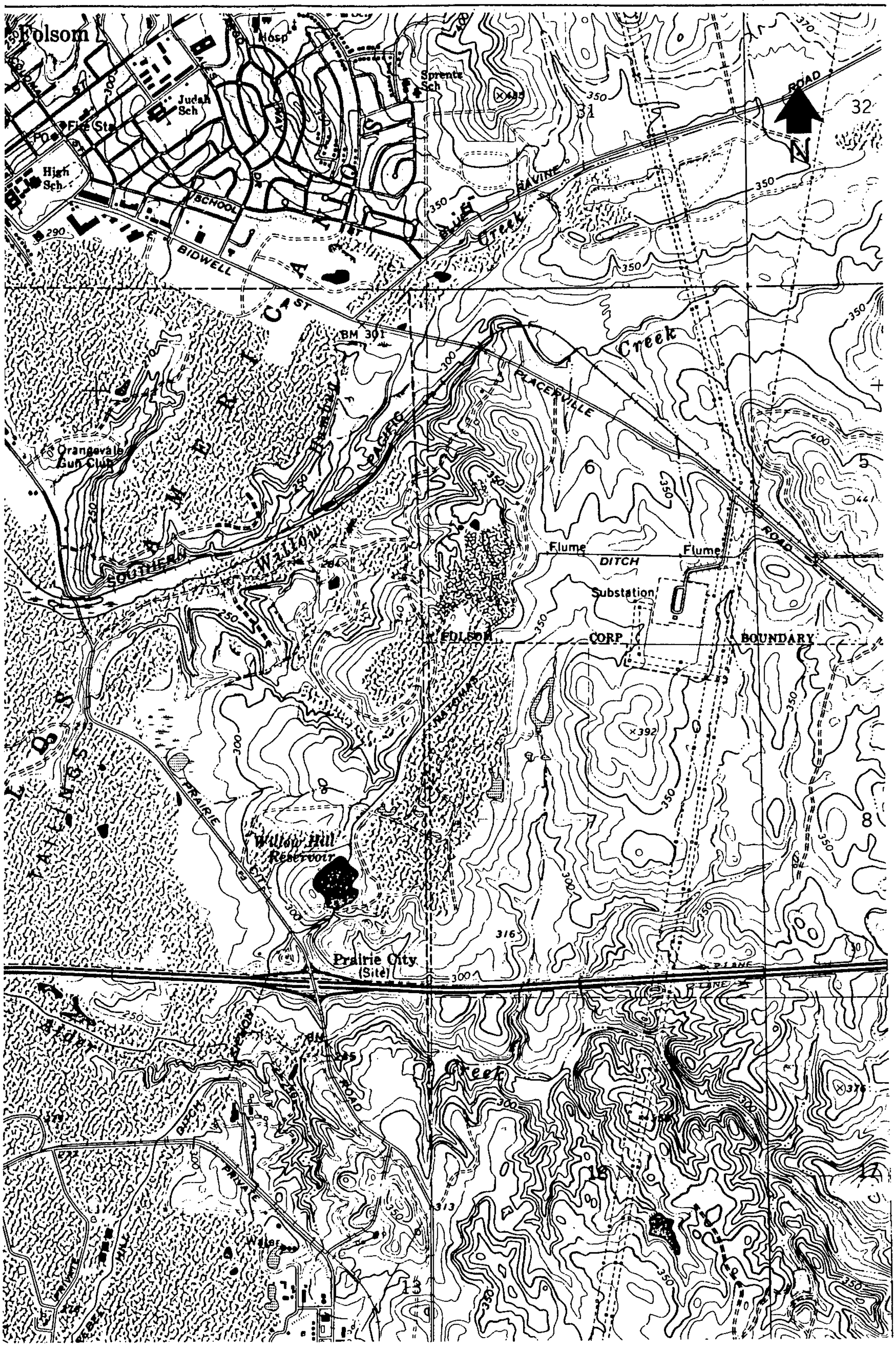
Mississipp

Source: USGS Topographical Map, Folsom Quadrangle



Mississippi Bar Location Map

FUGRO McClelland



3

FIGURE 3-18

residential, recreation, and wildlife habitat. Bike and equestrian trails travel through the bar and cross a portion of the Teichert operation.

3.1.6 YUBA RIVER DREDGE FIELDS

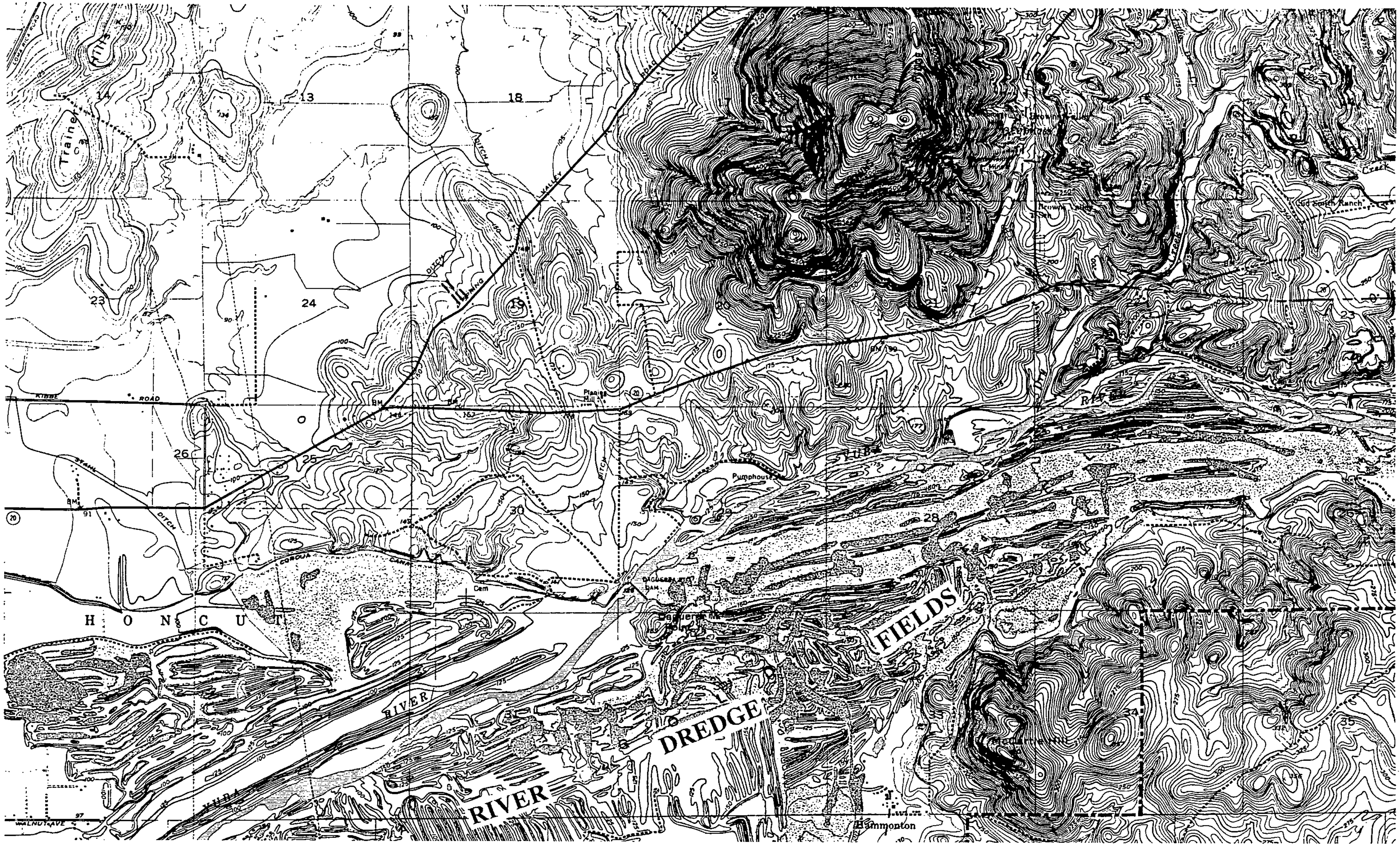
Located in eastern Yuba County immediately north of Beale Air Force Base, the Yuba River dredge fields consist of approximately 8,500 acres of dredge tailings on either side of the Yuba River. Figure 3-19 shows the location of the designated mineral resource zone. The land surface in the area, which has been disturbed as a result of historic gold mining operations, is characterized by discontinuous elongate tailings mounds or "windrows" with intervining ponds and riparian areas (Figure 3-20). Aggregate harvesting operations in the area excavate and process the tailings for use as construction materials.

The Yuba River MRZ-2 (Mineral Resource Zone 2; see Section 4.1, Land Use, and Figure 3-19) area consists of four types of deposits: 1) natural stream channel and floodplain alluvium, 2) hydraulic wash deposits from upstream monitor workings, 3) dredge tailings which are reworked natural alluvium and hydraulic wash deposits, and 4) recent stream channel alluvium in the present channel of the Yuba River (CDMG, 1988).

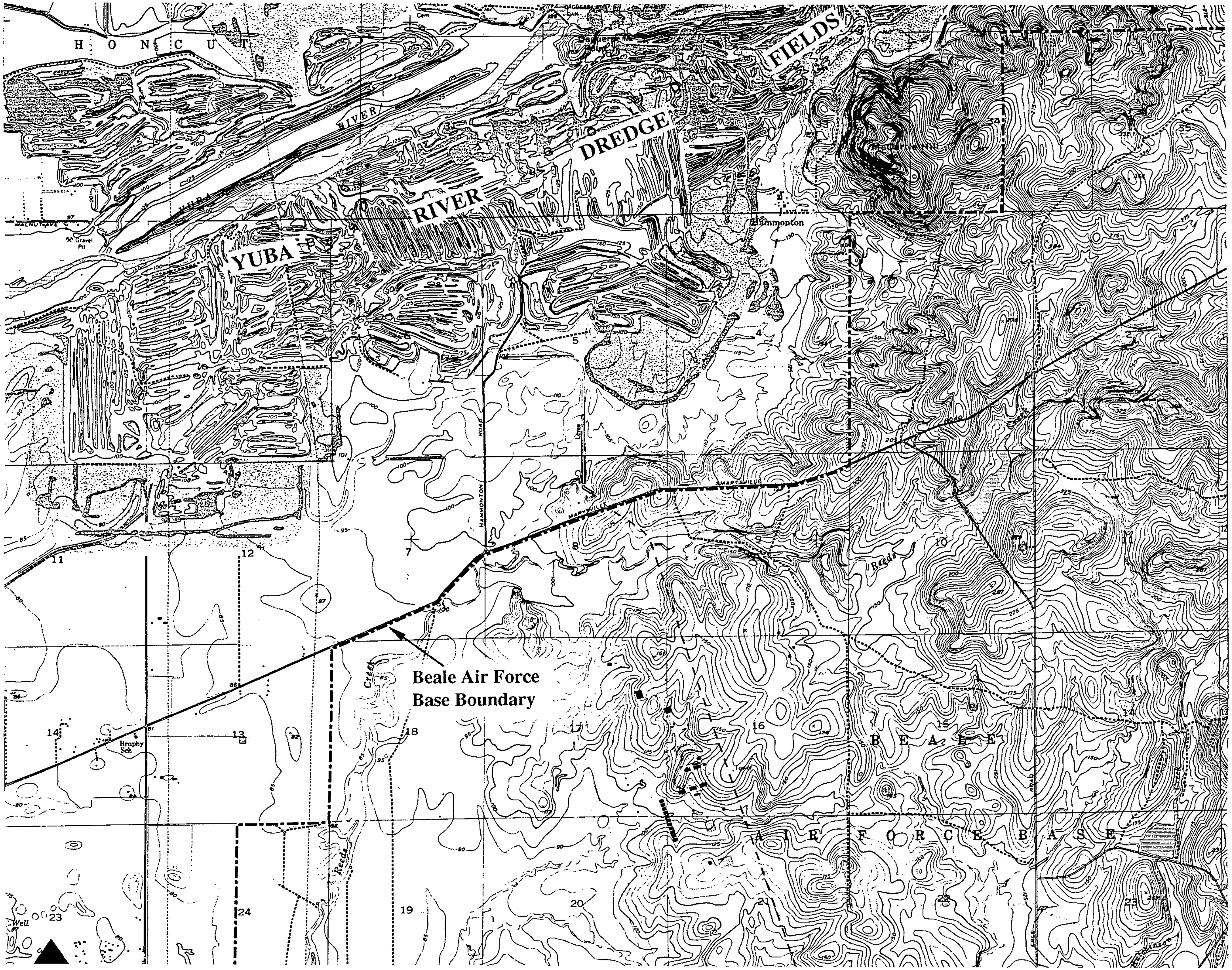
The natural alluvium was deposited from Tertiary to Recent times when the Yuba River carried large volumes of sand, gravel, and silt into the Central Valley. The abrupt decrease in gradient as the river entered the flat valley caused a decrease in stream transport energy which resulted in sediment deposition, and the creation of the floodplain. This is a normal fluvial transport process and occurs mainly during flood stages.

The hydraulic wash deposits were emplaced during the late 1800s, primarily from 1852, when hydraulic mining started in the upper Yuba River drainage basin, until 1893 when the Caminetti Act was passed creating the California Debris Commission. During this time Aubury estimated "...that aside from the natural sedimentation due to erosion, that nearly half a billion cubic yards of hydraulic tailings have been carried down yearly by the flood waters and deposited in the river valley" (Aubury, 1910). Originally, the Yuba River flowed in a deep "V" shaped gorge that may have been as deep as 75 feet. By the late 1890s, the Yuba River was choked with mining debris and the deep gorge had disappeared. Some natural sediments and probably also old hydraulic tailings continued to be transported to the lower Yuba river area from the time hydraulic mining was banned until 1941 when the Englebright Dam was completed.

In 1902, gold dredging began near the town of Hammonton and by 1910, 15 dredges were operating in the lower Yuba River (Aubury, 1910). The dredge tailing area has been dredged and re-dredged intermittently to the present time to progressively greater depths. This dredging of both the underlying natural alluvium and the hydraulic wash cover has produced a field of cobble windrows and linear pools of water. The windrows contain an enormous quantity of PCC (portland cement concrete)-grade aggregate.

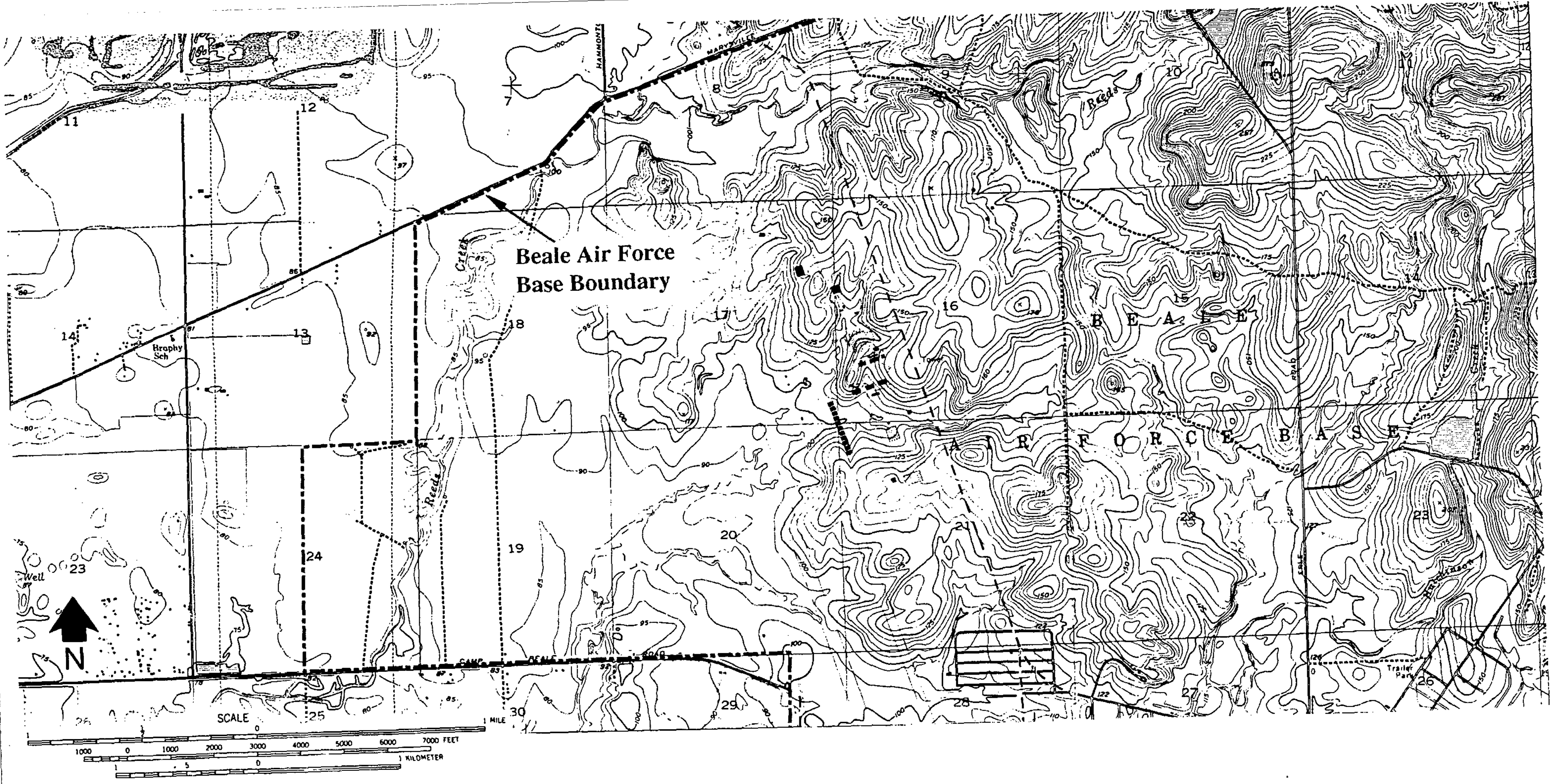


RIVER
DREDGE
FIELDS
H O N C U T
RIVER
YUBA
Pumphouse
Hammonton
Trainer
KIBBE ROAD
DITCH
Cem



H O N C U T
FIELDS
DREDGE
RIVER
YUBA
Hammonton
Beale Air Force
Base Boundary
B E A L E
A I R F O R C E B A S E
Brophy Sch
Gravel Pit

2



Yuba River Dredge Fields Location

FIGURE 3-20

Source: USGS Topographical Map, Brown's Valley Quadrangle

The active Yuba River channel within both the natural levees and man-made levees, contains unweathered gravels, sands, and silts. Since the completion of the Englebright Dam in 1941, only minor amounts of sand and gravel have been added from the tributaries below the dam.

Currently, the three largest companies which operate in the area are: Teichert, Baldwin, and Western Aggregate. Western has the largest operation and currently has access to 4,000 acres. Neighboring operations include a concrete batch plant and a precious metals dredging operation. The land surrounding the tailings are primarily in rural/agricultural uses. Beale AFB is visible to the south. The dredge tailings themselves are also used extensively for wildlife habitat.

3.2 AGGREGATE EXCAVATION, PROCESSING AND TRANSPORT

In an effort to identify potential environmental impacts, the following sections present hypothetical operating scenarios for the various alternative aggregate sources. The operating scenarios are grouped into three categories: 1) the Middle Fork sand and gravel operation; 2) quarry operations and 3) harvesting and processing operations distant to the dam site.

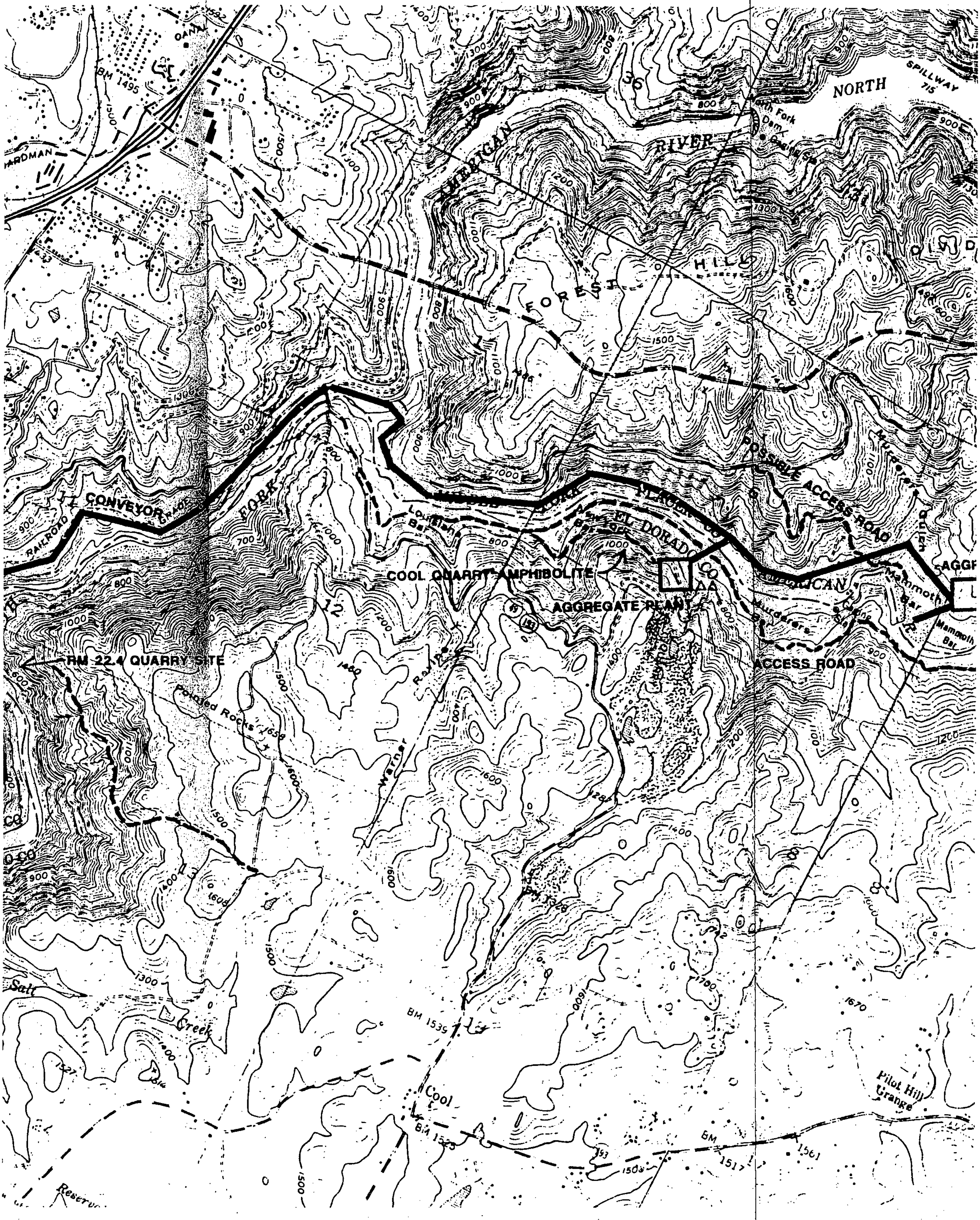
3.2.1 MIDDLE FORK SAND AND GRAVEL OPERATION

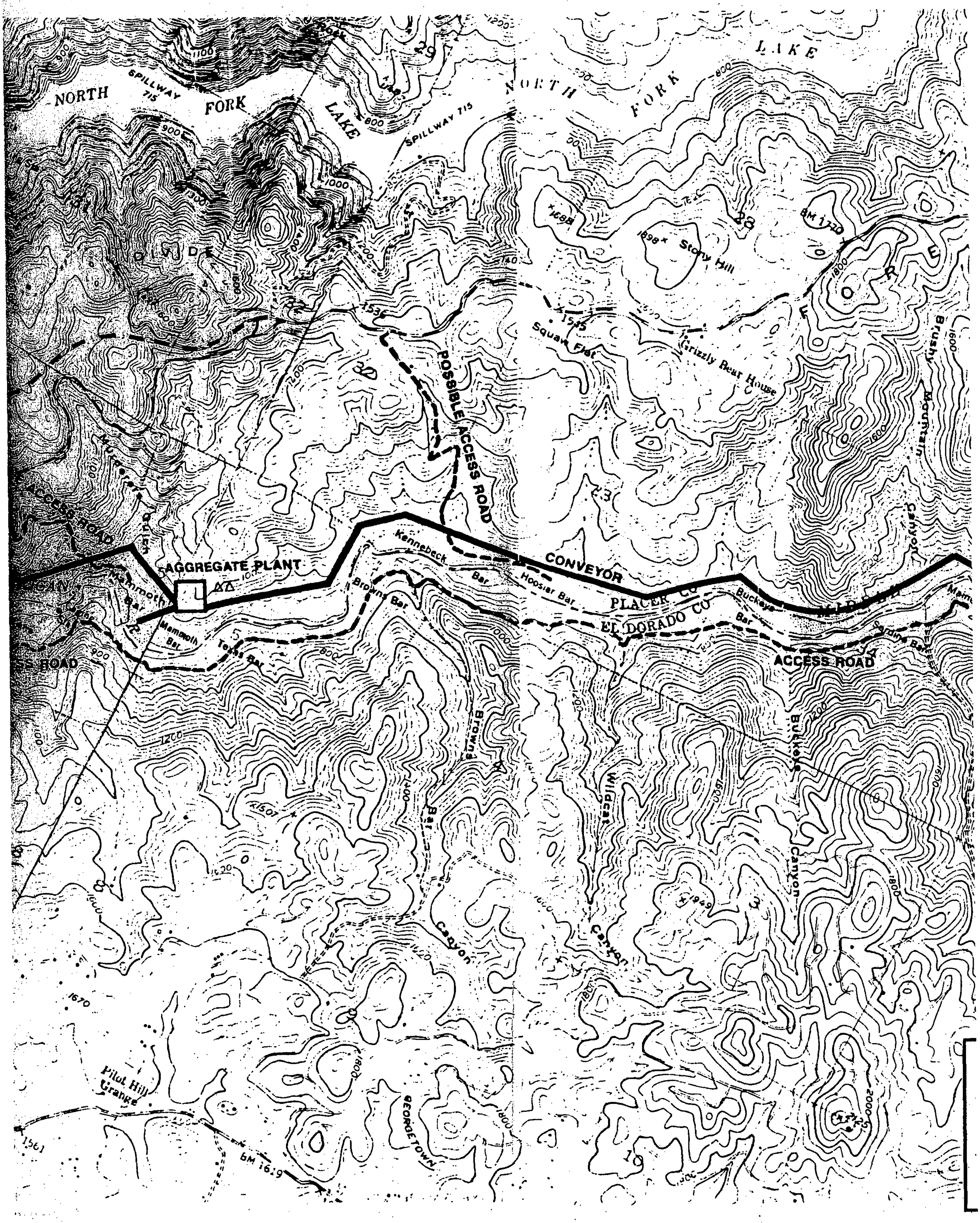
The following sections provide a generalized description of facilities required to extract, process and transport aggregate from the Middle Fork American River bars to an aggregate processing plant. The excavation and production of aggregate to satisfy the 6.76 million cubic yard demand for construction of the dam are expected to occur over a two to three year period. The facilities described below were determined based on information from existing aggregate mining and processing operations and on the estimated volume of materials necessary for dam construction. Various preliminary environmental considerations which may affect the location of facilities or the timing of certain activities are also indicated.

Implementation of a proposal to harvest sand and gravel along the Middle Fork would require development of a temporary aggregate extraction, conveyance and processing operation immediately adjacent the river. Figure 3-21 shows possible aggregate processing plant sites and conveyor routes. The operation would involve several hundred (300+) acres of state and federal lands extending from the Auburn Dam site upstream along the Middle Fork to Cherokee Bar. The area available for mining totals approximately 180 acres. The estimated yield of the ten bars is 8.6 million cubic yards of sand and gravel. Sand and gravel from Sardine Bar and the river channel may be extracted to supplement the aggregate supply if material from the exposed bars is not sufficient. Estimates indicate that an additional 1 million cubic yards of material are available from these supplemental sources.

Millertown
(Site)
AUBURN
CONVEYOR
NORTH
FORK
RM 22.4 QUARRY SITE
CONCRETE PLANT
AUBURN DAMSITE
PLACER CO
EL DORADO CO
FAIRGROUNDS
County Hospital
Courthouse
Salt
Creek
(PILOT HILL)
1761 I SE
SCALE 1:24000
OREGON BAR PLUTON QUARRY SITE
CONTOUR INTERVAL 20 FEET

2


NORTH
SPILLWAY
715
RIVER
North Fork Dam
AMERICAN
FOREST
HILL
36
CONVEYOR
RAILROAD
FORK
MIDDLE FORK
EL DORADO
PLACER
POSSIBLE ACCESS ROAD
COOL QUARRY AMPHIBOLITE
AGGREGATE PLANT
ACCESS ROAD
RM 22.4 QUARRY SITE
Pointed Rocks
Mammoth Bar
Murderers
Salt
Creek
BM 1495
BM 1535
BM 1525
BM 1517
Cool
Pilot Hill Grange


NORTH
SPILLWAY 715
FORK
LAKE
SPILLWAY 715
NORTH FORK LAKE
DIVIDE
Stony Hill
Squaw Flat
Grizzly Bear House
Brushy Mountain Canyon
POSSIBLE ACCESS ROAD
ACCESS ROAD
AGGREGATE PLANT
CONVEYOR
Kennebeck Bar
Hoosier Bar
PLACER CO
EL DORADO CO
Buckeye Bar
Browns Bar
Mammoth Bar
Texas Bar
ACCESS ROAD
Browns Bar Canyon
Wildcat Canyon
Buckeye Canyon
Pilot Hill Grange
GEORGETOWN

4

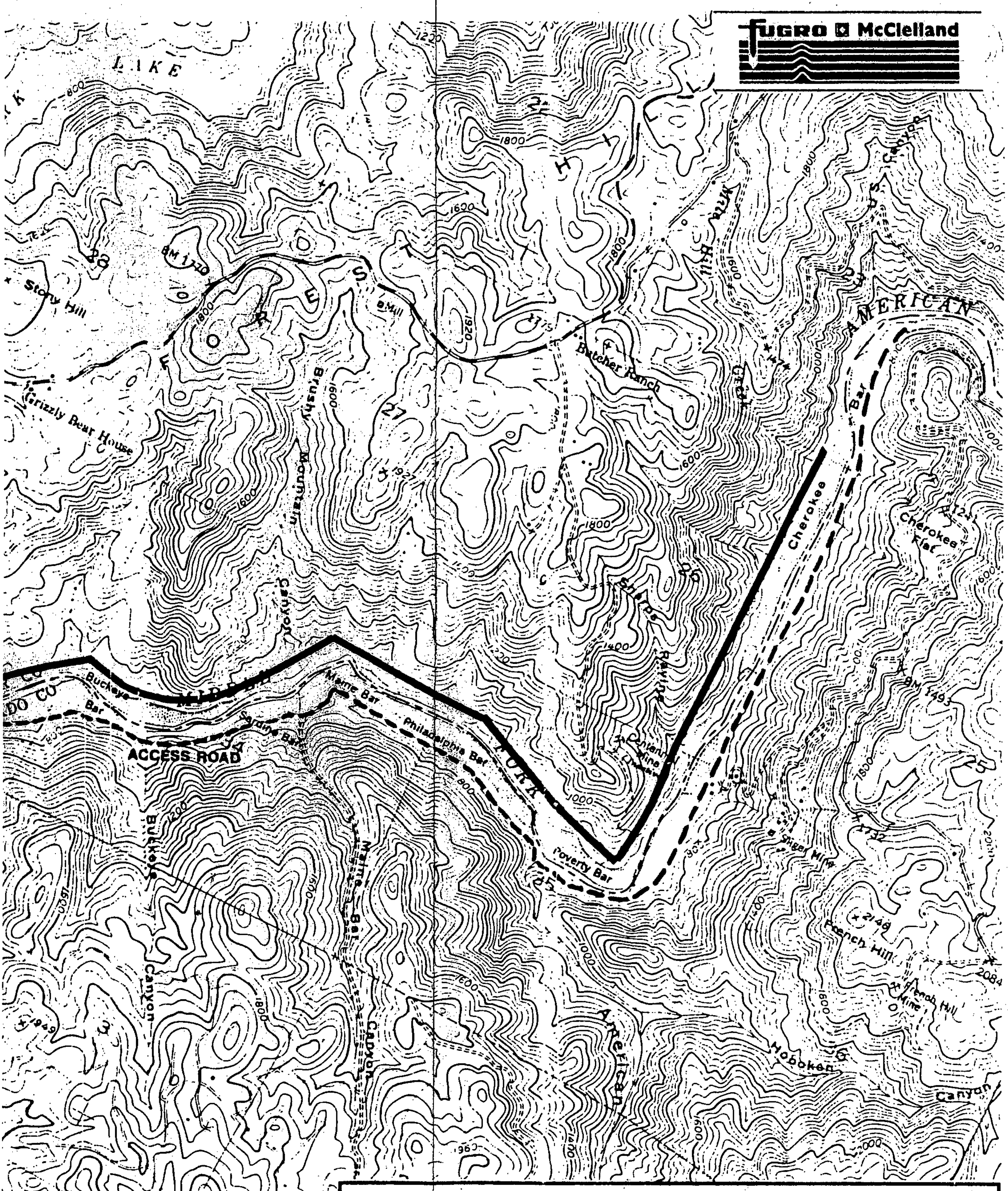


FIGURE 3-21

Possible Aggregate Plant Sites, Conveyor Routes and Access Roads

Source: U.S. Corps of Engineers, 1991, USGS Topographical Map, Auburn and Greenwood Quadrangles

The description of this alternative assumes that excavation of the bars would begin at Mammoth Bar and continue upstream to Cherokee Bar. Excavation activities would occur at two or three bars concurrently, depending upon the size of the bars being worked and the size of the excavation equipment. Sardine Bar would not be excavated unless the results of the Corps' Preliminary Engineering Design Study (PED) determine a need. The bars contain sand and gravel of various sizes. Initial estimates based on the gradation curves generated by USBR investigations indicate that approximately 23 percent of the aggregate will need to be crushed and all of it will need to be processed to be suitable for RCC aggregate as recommended by the American Concrete Institute (ACI).

Site Preparation

Access. Temporary roads would be required to provide access to the excavation and processing sites. These routes would connect to existing county or state roads (Figure 3-21) and would allow work crews with their accompanying equipment to access the 12-mile stretch of canyon bottom. Reconstruction of existing unimproved routes in the canyon would include widening to a minimum of 25 feet with grading and filling as necessary. A 15-20-foot right-of-way would be necessary to accommodate the primary conveyor which would ultimately run the 12-mile length of the canyon bottom (Figure 3-21). Improvements and construction of these routes would occur as excavation progressed upstream. Site access at the processing plant would need to be wide enough to allow two-way travel of employee vehicles. Roads to the processing plant would need to be developed prior to plant construction.

Aggregate Processing Site. A possible aggregate processing plant site has been indicted north of Mammoth Bar (Figure 3-21). Preparation of the site would include clearing of vegetation, and grading to level the site. The initial construction at the site would also include development of an interim surface drainage system including berms and channels to direct overland flow into settling ponds and away from stockpile or processing areas. If necessary, the drainage system may include a network of pipes to minimize groundwater seepage.

The plant's processing units would be arranged on-site to provide efficient processing. Materials would be transferred from one unit to the next by a system of conveyors. The plant structures would occupy 4 to 7 acres of the site (Hess, pers. comm., 1991). Additional acreage (5-10 acres) would be needed for processed material storage, vehicle parking, fuel storage, plant offices and main control building. All of these structures would be temporary.

Conveyor System. The primary conveyor would run sub-parallel to the river channel some distance above the high water mark (e.g., the 10-year flood level). Figure 3-21 shows a possible route along the north shore. The right-of-way for the conveyor would need to accommodate the conveyor itself (5-10 feet wide) and an access road (10-15 feet wide). The right-of-way would extend the 12-mile length of the river from Cherokee Bar to the dam site.

Portable secondary conveyor segments would be placed at various locations along the primary conveyor route. These secondary segments would be used to transport pitrun or partially processed material from the excavation sites to the primary conveyor which would direct the material to the aggregate processing plant. Both the primary and secondary conveyor segments would be portable and placed on-line as needed. Preparation of the conveyor right-of-way would also occur as needed. The conveyor system would be powered by several diesel generators.

Excavation Operations

Equipment. The specific excavation equipment has not been determined in detail. Due to the size and depth of the Middle Fork deposits, and the location of the aggregate, the Corps has indicated that draglines are the preferred type of equipment for excavation. Figure 3-22 shows a dragline excavation setup with a 2.5 cubic yard capacity bucket. Based upon the required production rate, three to four large draglines would be needed for excavation.

A portable track-mounted primary processing unit would also be stationed at each excavation site. As two bars may be excavated simultaneously, two or three primary processing units would be needed. The processing unit would consist of a hopper, primary screen, and jaw crusher. Portable secondary conveyors would transport material from the primary processing units to the primary conveyor system.

Other equipment would include a water truck for dust control and maintenance vehicles for the draglines, conveyors and primary processing plants. These vehicles would be located at or near the excavation site during work shifts. Front-end loaders and bulldozers would be necessary for material handling at the primary processing units and at the aggregate processing plant. Worker vehicles would be parked in designated areas near the excavation and processing sites.

Excavation Method. Excavation of material would begin at Mammoth Bar and proceed upstream to Cherokee Bar. One or two draglines would be set up at each bar, depending on available space. Excavation would proceed through-out the bar, down to near bedrock, until all useable material is exhausted (Figure 3-22). Boulders too large to crush would be left in place. The sand and gravel deposits would be scooped by draglines and directly loaded onto a secondary conveyor or dumped into a stockpile where it would be loaded onto a conveyor by an automatic feeding unit. To maintain the necessary level of extraction to feed the processing plant, it is likely that two bars would be worked simultaneously.

Due to annual flooding of the river bars, it is assumed that the bars would be accessible 8 to 10 months out of the year. Based on an excavation time line of two years, approximately 30,000 tons of aggregate must be produced each day during this period to provide an adequate supply of material for year-round concrete production. Assuming a 12-hour work day, approximately 2,500 tons of aggregate must be produced hourly.

Dragline in Operation at Mammoth Bar

Source: U S Bureau of Reclamation

FIGURE 3-22

Aggregate Processing Operations

Plant Facilities. Basic operations of a typical sand and gravel plant include crushing, washing and screening (sizing) of the unprocessed or pitrun material. Main processing facilities at the plant would include a wash plant, cone crusher and crushing facility. Various sized screens would be used within these structures to classify and separate the material by size. Conveyors would link the main structures, transporting material from one processing unit to the next, eventually directing finished product to a stockpile. Purchased electricity would likely provide the power source for the production plant.

Plant Operations. Material arriving at the processing plant would be dumped into a large surge pile which would serve as the raw material for the processing operation. The primary screening operation begins by segregating unprocessed material into three major fractions: sand, gravel, and cobble. Size classification of the materials is achieved by a variety of equipment, depending on the particle size involved.

Sand used for concrete is material having grain size varying from 3/16 inch to No. 200 mesh sieve. At the upper limit it grades imperceptibly into fine gravel and at the lower limit to silt. Gravel used for concrete is material varying from about 1-1/2 inches down to 3/16 inch. The maximum size may vary; it may be 3/4, 1-1/2 or occasionally 2-1/2 to 3 inches. Larger particles would be reduced in size by a combination of jaw and gyrator crushers. The gravel would go through several screening operations to be segregated into stockpiles by particle size.

Sand suitable for concrete is separated by passing material through successively smaller screens. The sand is washed to remove undesirable silts and clays, dewatered and conveyed to stockpiles. In a typical plant setup, the washing is done in what is called the "wet" side of the plant. There is little or no dust emission from this part of the plant because water spray bars are used at all transfer points and screens to wash off clay and silt. Processed aggregate is directed to storage piles until needed.

Water Supply and Use. Water is necessary for aggregate processing and dust control. Water use for the plant itself is not necessarily consumptive. It may be pumped from a series of shallow wells near the processing plant or the river itself. Once it has been used in the wash process, it would be directed to lined settling ponds. Depending on ambient conditions, some water used for dust control purposes may be lost to evaporation.

The amount of water used depends upon the pitrun gradation (relative distribution of particle sizes) and the cleanness requirements for the final product. Water use for the processing operation is estimated at 10 gallons/minute per ton/hour. If the plant produces 1,000 tons/hour, the water use rate would roughly be 7,200 gallons per day. Assuming that the aggregate plant would operate six days per week, the annual water usage for processing would be approximately 2.0 million gallons. Actual consumption would be much less due to extensive recycling of water at the plant.

Stockpiles. According to the Corps' Special Aggregate Report, approximately 40 acres would be needed to store processed aggregate. Stockpiles would be located at various stations along the river channel between the aggregate plant and the dam site. Additional storage area would be required at the plant site itself to accommodate pitrun and processed aggregate. This aspect of the proposal requires further investigation.

Hours of Operation. The plant would operate for 12 hours per day, six days per week. It is also assumed that excavation would occur 8 to 10 months of the year and slow down or cease during the wet winter months. The aggregate processing plant would operate year round. This would necessitate at least a two-month supply of pitrun material at the processing site.

Security. The processing plant would be fenced with gated entries. "No Trespassing" signs would be posted as necessary. Access to the facilities during operation hours would be restricted to employees, authorized visitors, and truck drivers. During off hours, access would be restricted to management personnel or authorized employees.

Employees. The contractor selected to perform the work would be responsible for hiring and assigning employees to the project. Based on the average number of employees at similar sized operations, approximately 20 to 25 individuals would be needed to handle the clerical, management and operations of the plant and excavation.

Environmental Considerations

Dust Emissions. Little dust would be emitted in the harvesting process due to the relatively high moisture content of the freshly mined material. The access routes would be unpaved and dust would be generated by traffic along these routes. A water truck would be used along access routes to minimize dust generation. Similarly, particulate emissions from processing and conveyance systems would be minimized using wet methods. Dust generated by crushing, screening, conveying and dumping of crushed rock would be controlled through a combination of baghouse filters and wet methods. Large stockpiles of aggregate can become a source of dust when the wind blows strong enough for the material to dry and become airborne.

Waste Material Disposal. Excavation and processing of pitrun material will result in the generation of waste fines or materials too small for use in concrete manufacture. Exploration results indicate 8.2 percent of the 8.6 million cubic yards available in the bars occurs as fine-grained material. This translates to over 700,000 cubic yards of silt and clay-sized particles. A portion of this material may be used in concrete manufacture. The rest would be removed from the flood plain to minimize water quality impacts.

It may be feasible to collect these materials and use them for reclamation of access roads and conveyor routes, or to reclaim other "scars" caused by excavation or quarry operations. Although reclamation activities would be part of the ongoing process, most roads would be

needed throughout the entire excavation period and reclamation of these areas would not occur until near the end of such activities. Use of fines for reclamation would therefore also create the need for storage of this material, in addition to pitrun and processed aggregate storage. Because storage area is not readily available along the canyon bottom, off-site disposal of the waste fines would be a necessary consideration.

Water Quality Control. As with any mining activity in close proximity to a waterway, a potential for turbid discharges would exist. In-stream excavation would be avoided to the extent possible. A series of settling ponds would be developed to allow for settling of fines during excavation and processing. The turbid water would infiltrate into the groundwater with the residual fines eventually forming a coating covering the bottom of the ponds. Periodic dredging and removal of these excess fines from the floodplain would be necessary to preclude turbid discharges during flood stages. Because the amount of space available for such ponds is limited, flocculating agents may be used to enhance the settling process. Also, mining and processing activities within the floodplain of the river would cease during the rainy season when inundation and erosion potential is highest.

Much of the processing and conveyance facilities would be electrically powered by diesel-fueled generators. Excavation and loading equipment would also be diesel-fueled. The potential for hazardous material releases would be minimized through maintaining fuel storage and refueling facilities out of inundation zones, and by development of operating procedures designed to minimize hazardous material incidents.

3.2.2 QUARRY OPERATIONS

Preliminary review of potential aggregate deposits (see Section 1.0) indicated two potential quarry-supplied sources of material in the vicinity of the dam site:

- **Old Cool Quarry (Spreckles Limestone and Aggregate)** - A currently operating commercial quarry located in the canyon of the Middle Fork, approximately 5 miles upstream from the dam site.

- **Cool Quarry Amphibolite** - A quarriable body of metamorphic rock located immediately west (downstream) of the existing Cool Quarry (Spreckles).

The following sections describe hypothetical quarry operations and outline planning and development considerations for establishing a new quarry operation in the American River canyon. The operational parameters of the Old Cool Quarry are reviewed and compared with those of the proposed Cool Quarry Amphibolite.

Planning and Development Considerations

While preliminary examination indicates the material at the proposed Cool Quarry Amphibolite is suitable for use as concrete aggregate, additional work is still necessary to insure the material meets necessary requirements. Essential properties, apart from considerations of divisional planes, jointing, bedding, or cleavage, are those associated with strength and durability. The material must not only be able to bear the stresses it will be subjected to in the dam, but also must be able to do so for an indefinite period of time. The entire materials study and design process can take up to four years (Hess, pers. comm., 1991).

To further determine the feasibility of the proposed quarry site, an expanded geological survey will be necessary. Previous sampling by USBR included 7 drill holes for a total of 1,259 feet of core. An expanded survey should define the dimensions of the deposit which in turn will dictate the operations plan for the running of the quarry. Necessary considerations in any future surveys are:

- The removal and disposition of overburden;
- Reclamation Plans;
- Avoidance of areas poorer quality stone;
- Quarrying process - including maximum length and height of quarry face;
- Transport considerations including truck haul routes, as well as conveyor and aggregate plant positioning.

If the Cool Quarry Amphibolite becomes the preferred alternative, preparation of a reclamation plan will be necessary. Appendix 6.1 contains the California Surface Mining and Reclamation Act of 1975 which specifies state reclamation requirements.

Aggregate Processing and Transport Considerations

Old Cool Quarry. At the Old Cool quarry, the current mining, processing and stockpiling activities are carried out in the upper portions of the property, near Highway 49 (Figures 3-14 and 3-15). Topsoil is stockpiled while contaminated rock (overburden) is used to backfill existing excavations left over from previous quarry operations. These large excavations are also used as settling ponds for wash water.

Transport considerations from the Old Cool Quarry would be similar to those of the Cool Quarry Amphibolite (see below). The close proximity of both the Old Cool Quarry and the Cool Quarry Amphibolite to the dam site minimizes the difficulty associated with conveyance of material via off-highway trucks or conveyors. While truck transport may not

be the most efficient method of conveyance, existing roads could be improved or new roads constructed to transport material down to river level for stockpiling or immediate conveyance to the dam site vicinity. Alternatively, transportable conveyors could be configured in such a way so as to minimize handling of the aggregate material.

Regardless of the transport mode, some provision will need to be made for crossing of SR 49. Transport of aggregate from Old Cool Quarry to the dam site will require crossing the highway at some point. SR 49 will need to be rerouted during the operation or some type of over/underpass will need to be constructed. The over/underpass could either serve trucks or a conveyor system.

Cool Quarry Amphibolite. Aggregate plant and transport issues must be resolved at the planning and development stage. Conservation of energy in any mine operation is the key to efficiency and a chief consideration is positioning of the aggregate processing facilities. Ideally, processing should occur as close as possible and downhill from the mining area. In addition to the 40 plus acres necessary for stockpiling finished product, five to ten acres of level ground are necessary to site a 1,000-ton-per-hour processing facility.

In their Special Aggregate Report, the Corps indicated a possible site located adjacent the Middle Fork near the Old Cool Quarry (Figure 3-21). Aerial photos taken in 1976 indicate a small processing facility occupied the 10-20 acre site at one time. Currently, El Dorado County stockpiles minor amounts of roadbase material in the area.

Quarry run material could be conveyed or trucked from the working face of the quarry to the processing plant. Because of its variable particle size, quarry run material accelerates wear and tear on transport systems and is more difficult to handle than processed material. At the processing plant material would be crushed, washed and separated into sizes according to design specifications. Processed material would be trucked or conveyed from stockpiles to a concrete batch plant located near the dam site (Figure 3-21).

The water supply for aggregate washing could be obtained from either the river or groundwater. Wash water would need to be treated through a series of settling ponds before reuse or discharge to the river. On-site drainage provisions would need to be made so as not to allow untreated stormwater runoff to flow directly into the river.

Quarry Operations

Amount of Overburden/Stockpile Area. The first phase in the commencement of quarry operations is removal of overburden. Overburden is material of any nature that overlies a deposit of useful material. The overburden removal process requires stripping of soil and vegetative matter to expose the target material. The surficial soil material is usually stockpiled for later use in reclamation. Typical ratios of overburden to ore (stone) for new quarries in the Sierra are 1 cubic yard of overburden per 2 cubic yards of ore (Bartley, pers.

comm., 1991). Overburden thickness at the Old Cool Quarry ranges from 10 to 80 feet in thickness.

In the absence of site-specific information, and for the sake of impact assessment, a rough estimate of overburden volume can be attained using the ratio above (overburden:ore = 1:2) and the amount of aggregate material necessary for the project (approximately 5 million cubic yards of solid rock). The resultant 2.5 million cubic yards of overburden would form a 60-foot tall conical-shaped pile (volume = Ah/3) covering an area of 75 acres.

The crude estimate shows that a considerable amount of material will need to be stockpiled in an area somewhere near the quarry site. Similarly, considerable area will be necessary to stockpile finished product. Some storage space is available between the quarry and the dam site on the south side of the river (Figure 3-21). In part due to its location and long history of production, the Old Cool Quarry has storage space available near the existing quarry operation. The current operation deposits overburden in large pits excavated by previous operations.

Mining Methodology. The operation stage of a new open pit quarry would commence with removal of the surficial soil mantle. Bulldozers and scrapers are typically employed to excavate the soil and transport it to a stockpile area. Next, weathered stone material is removed down past the zone of weathering to fresh unaltered stone. This stage of overburden removal usually requires drilling and blasting. The contaminated stone is removed using a large shovel or bulldozers and hauled by trucks to stockpile areas or to a conveyor.

Once minable material is reached, actual quarrying of the rock commences with drilling, blasting and removal. As production continues benches are formed at successively deeper levels. Ultimately, the quarry pit will be similar in configuration to an upside-down wedding cake with a series of benches approximately 20 feet wide and 40 feet high. Geotechnical and safety considerations will dictate the ultimate bench widths and face heights.

Depending on the size of the operation, the two machines which occupy major roles in the stripping of over burden and the mining of material are the shovel and the dragline. Also ancillary machines such as bulldozers, front-end loaders and scrapers are employed. Material is transported from the working face by pit trucks to the primary jaw crusher for initial sizing and from there to other secondary crushing, screening and washing facilities. Currently, the Spreckles operation at the Old Cool Quarry has a 600 ton per hour processing capability that operates intermittently.

Magnitude of the Quarry Operation. Impacts associated with a given quarry operation are directly related to the amount and rate of production. Preliminary estimates by the Corps indicates 6.76 million cubic yards of aggregate will be necessary to produce 5 million yards of concrete. If dam construction is to be complete within two years, the quarry operation would need to be designed to meet the construction schedule. Extending construction into

a third year or stockpiling of aggregate prior to the start of construction would also allow production to proceed at a slower rate, thus helping to minimize potential impacts.

The production rate has implications for the size and amount of processing and transport equipment as well as the rate of drilling and the magnitude and frequency of blasting. Quarry and processing operations would likely need to proceed continuously with drilling and shooting occurring around the clock in different parts of the quarry. Lighting would need to be installed for night operation. Conveyers and/or pit trucks could run 20 to 24 hours per day. Except for maintenance and breakdown periods, the aggregate processing facility would operate continuously.

Similar to the estimates given above for overburden amounts, the ultimate size of the quarry excavation can be roughly approximated by making the following assumptions:

- 5.2 million cubic yards of ore (6.76 divided by 1.3 volume factor)
- 2.6 million cubic yards of overburden
- 1:2 pit slope (40-foot faces and 20-foot benches)
- conical shaped pit ($V=3.14r^2h/3$)

The calculations assume that the pit would be in the form of a right circular cone with a volume of 7.8 million cubic yards and 1:2 (horizontal:vertical) pit slope. The resultant excavation would form a 1,000-foot deep hole covering 16 acres at the surface. In reality, the excavation would likely be much shallower and cover much more surficial area. Planimetric measurements indicate approximately 40 acres are available for the new quarry.

3.2.3 SAND AND GRAVEL OPERATIONS DISTANT TO THE DAM SITE

Preliminary review of potential aggregate deposits (Section 1.0) indicated the following aggregate sources distant to the dam site:

- **Mississippi Bar on the Lower American River** - An extensive deposit of sand and gravel located approximately 18 miles from the dam site near Lake Natoma.
- **Bear River and Chevreaux Quarry** - Fluvial deposits of sand and gravel and quarriable rock located on the Bear River, along highway 49, approximately 11 transport miles north of the dam site.
- **Yuba River Dredge Fields** - Extensive deposits of sand and gravel located approximately 40 miles north of the on the Yuba River north of Beale AFB.

Mississippi Bar

At the Mississippi Bar deposit, a large front-end loader is used to excavate and dredge tailings from the site and load them into 25-ton trucks. The trucks transport the material to the neighboring Teichert processing facility where the aggregate is processed in plant similar to that described later in this section (see Yuba River Dredge Fields - "Wet" Plant). The Teichert plant's capacity is approximately 300 tons per hour with reserves expected to last until the year 2000.

Bear River and Chevreaux Quarry

The Chevreaux Company mines two types of aggregate sources within the property: quartzose alluvium and metamorphic basement rock. Both sources are used to make suitable Portland cement concrete grade aggregate. The alluvial deposit covers roughly 457 acres and the exposed metamorphic rock covers an additional 697 acres (Dupras, 1983).

The alluvial aggregate in the study area (Dupras, 1983) reached a maximum size of six inches and grades into a fine sand. Most of the coarse aggregate is concentrated at the Bear River inlet into Lake Combie. The aggregate size decreases downstream in a two-mile stretch along Lake Combie and becomes predominantly sand-sized directly behind Lake Combie Dam.

A dragline extracts the aggregate from the lake and the river shore during the winter months. Since the aggregate is roughly classified by water action along Lake Combie and at the mouth of the Bear River, rough sizing control is provided for the pitrun material by periodically changing the excavation sites.

Crushed Rock. The Chevreaux crushed rock aggregate is processed from altered dark greenstone breccia. The Joe Chevreaux Company mines the greenstone by drilling and blasting. Processing the rock for aggregate is accomplished through a series of crushers, screens, and classifiers. After crushing, grading specifications are met by screening and classifying the material. There is no waste material.

Yuba River Dredge Fields

While three operations currently exist in the Yuba River area, this section describes Western Aggregate's operation. Because of similar conditions and materials, it can be inferred that other companies operate in a similar fashion.

Finished Products and Operating Schedule. Western Aggregate produces sand, round rock, and recently added a crushing facility to reach a different segment of the market. Products are adapted to accommodate market conditions. In 1990 sales reached 1.2 million tons; overall production was 1.4 million tons. Plant production rate with existing structures is 1,000 tons/hour with plans to expand to 3,000 tons/hour. Electricity is supplied from PG&E

at a cost of 24 to 30 cents per ton of material produced. Currently, processing facilities are in temporary locations as the company plans to move the structures closer to the sources of raw material.

The Western Aggregate facility currently operates 8 hours per day and could add a second shift. In 1990, during peak periods, production was expanded to 16 to 18 hours per day. There are a total of 22 employees at the facility; 13 plant operators/truck drivers, 9 administrative/management personnel. Currently the company has access to 4000 acres and estimates reserves of 600-700 years at current production rates. Plans for a rail line through Beale AFB are definite, although the route has not been determined. Three alternate routes are being considered.

"Wet" Plant. The pitrun material is fed into the wash plant and the aggregate is "washed." The wash plant also provides initial screening of the material which is sorted onto different conveyors according to size. Conveyor belts are used between units to move material from one process to the next. Generally, the conveyors operate better if the material conveyed is of uniform size; pitrun material which consists of different sized particles causes non-uniform loading on the conveyor belts. The conveyors at the Western Aggregate site are portable sections, up to 100 feet long, pieced together. Cranes or loaders are used to move sections.

The wash plant processes approximately 500 tons per hour. Some of the water is recycled through the plant. The remainder is piped, together with the fines, to settling ponds. The water percolates into the tailings leaving behind the fines which may be reclaimed and sold as fill sand. Water quality in the Yuba River is monitored on a regular basis. Occasionally, high levels of silica are detected.

In general, for every 100 tons of product processed in the "wet" plant, about 12 tons of waste fines are produced (12 percent). Much of the waste is reclaimed as fill sand and mixed with road base as the clay content makes the mixture more cohesive. Depending on market conditions, Western Aggregate may reclaim 7-8 percent of the fines.

"Dry" Plant. First the material is sent to a jaw crusher where metal plates crush rock greater that 4 inches in diameter. The maximum capacity of the crusher is 600-700 tons per hour which actually pops the rock at its natural seams. The 4-inch minus material is then dumped into a surge pile (stockpile) where it awaits transport by conveyor to the crushing facility.

The crushing facility consists of a cone crusher (similar to a mortar and pestle), a barmat crusher and classifying screens. The classifier consists of a series of screens which separate particles according to size and directs them via conveyor to various stockpiles of finished product. A main control center located near the crusher allows the operator to monitor gages and crusher funnels to make sure the facility is functioning properly. Once through the cone crusher, material is either sent to stockpiles or the barmat crusher which crushes

particles to fines or "crusher dust." The dust is sold to paving companies as filler material for asphalt and roadbase. Dust control methods at the plant include a combination of water sprays and vacuum systems.

4.0 ENVIRONMENTAL IMPACT ANALYSIS

Thresholds of significance utilized for environmental issues analyzed in this report were developed based on CEQA Guidelines Appendix G, local/regional plans and ordinances and consultations with representatives from various governmental agencies. Criteria pertinent to individual environmental issue areas (biological, water quality, etc.) are presented in the relevant sections.

Different categories of impact significance require various administrative actions by the decision-makers at the time a project is approved. In the analysis to follow, several impact evaluation distinctions have been made. The different types of impacts that have been distinguished include:

Class I — Significant adverse **impacts which cannot be mitigated or avoided.** A significant unmitigable impact is a problem for which a solution has not been formulated due either to the limits of technical and/or scientific knowledge or infeasibility from a technical, economic, and/or political basis. These impacts require decision-makers to make findings of overriding considerations if the project is approved.

Class II — Significant adverse environmental **impacts that can be feasibly mitigated or avoided.** In these cases, the consequences of a project are considered sufficiently serious that some form of mitigation planning is needed. This mitigation can involve modifications to the project, changing the project design to avoid conflicts with environmental values, or performing data collection procedures prior to construction (such as archaeological salvage programs). Under section 15091 of CEQA, decision-makers are required to make findings that impacts have been mitigated as completely as possible to approve a project with Class II impacts.

Class III — Adverse project **impacts found not to be significant.** Adverse impacts describe the consequences of a project that are not sufficiently disruptive to require mitigation measures. Minor changes in the environment that have no serious consequences on the abundance or diversity of plant or animal life, for example, are classified as adverse but not significant. Minor changes in traffic flow, aesthetics, or air quality are other examples of insignificant impacts. There are factual tests recommended in the Appendices to CEQA that aid in this classification process.

Class IV — Beneficial project impacts.

The following sections of this report serve to disclose potential impacts resulting from utilization of any of the six identified aggregate sources for the Auburn Dam. Impacts are identified on the basis of the tentative project descriptions developed in Section 3.0.

4.1 LAND USE

Six aggregate source sites are under consideration, three in the canyon of the Middle Fork of the American River and three located at distant sites in Sacramento, Placer and Yuba Counties.

4.1.1 EXISTING CONDITIONS

Land Management

Three of the potential aggregate source sites under consideration are located in the canyon of the Middle Fork of the American River, between one and twelve miles from its confluence with the North Fork. These are the Cool Quarry Amphibolite, the Old Cool Quarry and the Middle Fork Sand and Gravel Deposits (refer to Figures 3-1 and 3-14). These existing and potential quarry sites are within the 42,000-acre Auburn Reservoir Project controlled by the U.S. Bureau of Reclamation (USBR) (Draft American River Watershed Investigation California Feasibility Report, DEIS/DEIR, April 1991, page 14-15). Private inholdings within the project area are administered by Placer and El Dorado Counties. The California Department of Parks and Recreation (CDP&R) manages public lands within this area which are designated as a State Recreation Area (SRA) under a contract with the USBR.

Land ownership in the Auburn Reservoir Project segment is 84 percent federal and 16 percent private. Federal lands acquired or withdrawn by the Bureau of Reclamation in the segment are managed for recreation by California Department of Parks and Recreation, which operates under an interim agreement initiated in 1977 and renewed annually. CDP&R developed a General Plan for the Auburn Project in 1978 under the assumption that Auburn Dam would be built as originally planned. Because of this, there has been very little development in the area to support recreation. Land use in the segment is primarily recreational, with minimal mining and residential inholdings (USDI, BLM, National Recreation Area Feasibility Study, Final, September 1990; page 25).

State Recreation Areas are established to help meet the non-neighborhood recreation needs of the public. Although the main emphasis is on outdoor recreation, the state's role is not restricted to that purpose. Lands are selected specifically for recreation purposes, for their ability to serve recreational needs on a large scale, and for the ability of their resources to withstand heavy visitor use. In SRAs, the recreation potential is the primary resource, with natural or cultural values supporting and enhancing the recreational setting. Planning and resource management activities are aimed at providing optimum recreation opportunities, in both quality and quantity. In planning and developing facilities in state recreation units, the precautions necessary in other classifications to protect the integrity to primary resources and values do not apply to the same degree. Protective standards have a different emphasis because the primary values of state recreation units are recreational opportunities rather

than natural features (USDI, BLM, National Recreation Area Feasibility Study, Final, September 1990).

County Land Use Plans

El Dorado County. The El Dorado County General Plan incorporates a number of community plans. The Cool-Pilot Hill Area Plan covers the area in which the Cool Quarry, the Cool Quarry Amphibolite and a portion of the Middle Fork bars aggregate sites are located. The Greenwood Area Plan is adjacent and to the east of portions of the Middle Fork Gravel Bar sites. El Dorado County is currently updating its General Plan; however, it is too early in the update process to determine whether management issues related to the Middle Fork will be addressed (Abramson, pers. comm., 1991). Since the majority of the properties influenced by the river are public lands, the current General Plan does not address these types of issues.

In 1988, voters in El Dorado County passed Measure A which requires permanent buffer zones between residential and urban uses and mining uses. This ordinance (Section 17.14.095 of the El Dorado County Code) applies to open pit and strip mining and requires that the "... boundaries of the proposed project for open pit mining or strip mining shall be greater than a linear distance of 10,000 feet from any existing residential use, hospital use, church use, or school use including but not limited to nursery or day care uses or any residential, hospital, church or school use as designated in the El Dorado County General Plan or any community or specific plan, or as permitted by the zoning code of El Dorado County." This ordinance applies only to projects which would require a discretionary permit from the El Dorado County Board of Supervisors. It would not apply to publicly held lands within the State Recreation Area.

Placer County. Placer County is currently updating its General Plan. Completion is expected within three years. A majority of the land found within the study area in Placer County is federally administered. Therefore, Placer County doesn't have a site-specific recreation management plan for lands within the study area. Under the Recreation Element of the 1971 General Plan, recreation use potential and environmental impacts were assessed by establishing a land classification system. Those federal and private lands found along the North and Middle Forks were classified as Class V - Primitive Area. The characteristics found in Primitive Areas were defined as:

> *...those lands that are extensively natural, wild and undeveloped, with a setting removed from the sights, sound, and smells of civilization. The area must be large enough and isolated as to give the user the feeling that they are enjoying a wilderness experience. Class V lands are those lands above 7,000 feet in elevation as well as all lands over 40% in slope.*

The primitive classification still applies to the Auburn project lands. Recommended recreation activities for lands now included within the Auburn project area were limited to

those that could be pursued without benefit of road access. The plan also recommended against the developments of permanent habitation or recreation facilities. Development of trail systems were found to be acceptable in the American River Canyon (USDI, BLM, National Recreation Area Feasibility Study, Final, September 1990).

Existing and Planned Land Uses

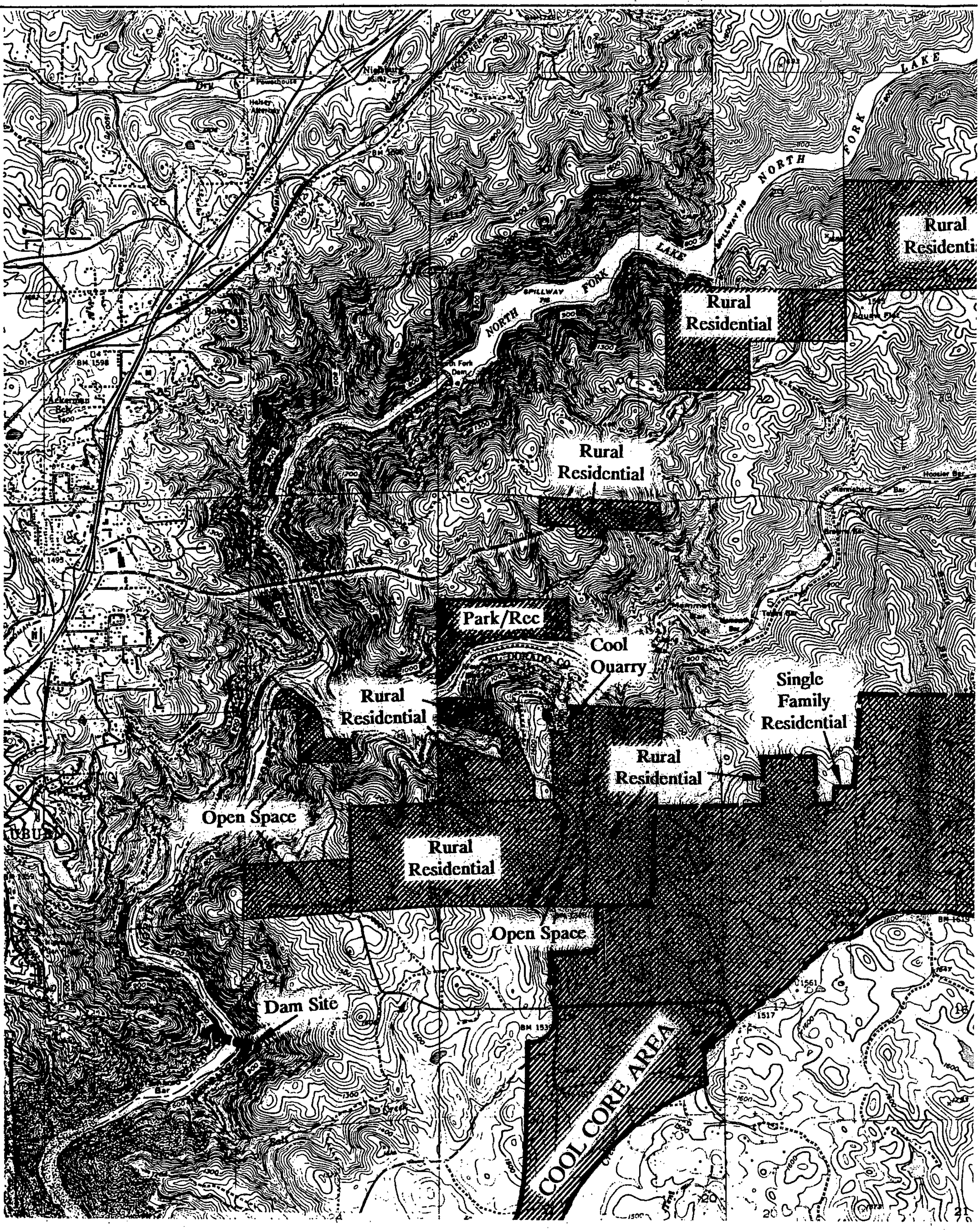
Middle Fork Sand and Gravel Deposits. The Middle Fork Bars are a series of 10 sand and gravel bars located along a 7-mile reach of the Middle Fork starting approximately 5 miles upstream from the dam site. The bars extend upstream from Mammoth Bar to the upper end at Cherokee Bar (Figure 3-14). Adjacent land uses along this reach of the river is open space and recreation. Canyon sides are steep in most places, making development impractical in this area. Most land is under the jurisdiction of the State Recreation Area; however, there are a few private inholdings immediately adjacent to the river on the Placer County side of the river in this area (Figure 4.1-1).

The Placer County General Plan designates the properties closest to the river as "Parks and Organized Recreation." Farther up on the ridge, where the properties are designated "Rural Residential," there is a potential for development densities of 0.2 to one dwelling unit per acre.

Old Cool Quarry. The Old Cool Quarry is located in El Dorado County approximately 7,000 feet north of the community of Cool (Figure 4.1-1). The Auburn Lake Trails residential development is 2,000 feet southeast of the quarry. Residences along Westville Trail, Shirt Tail Trail and Wild Cat Trail overlook the quarry (refer to Figure 4.9-2). Noise from the existing operations is audible at residences which are not blocked by topography. This quarry is an existing permitted operation in El Dorado County and was in operation prior to the enactment of Measure A. The current operators of the quarry, Spreckles Limestone and Aggregate, lease the property from USBR on a biannual basis.

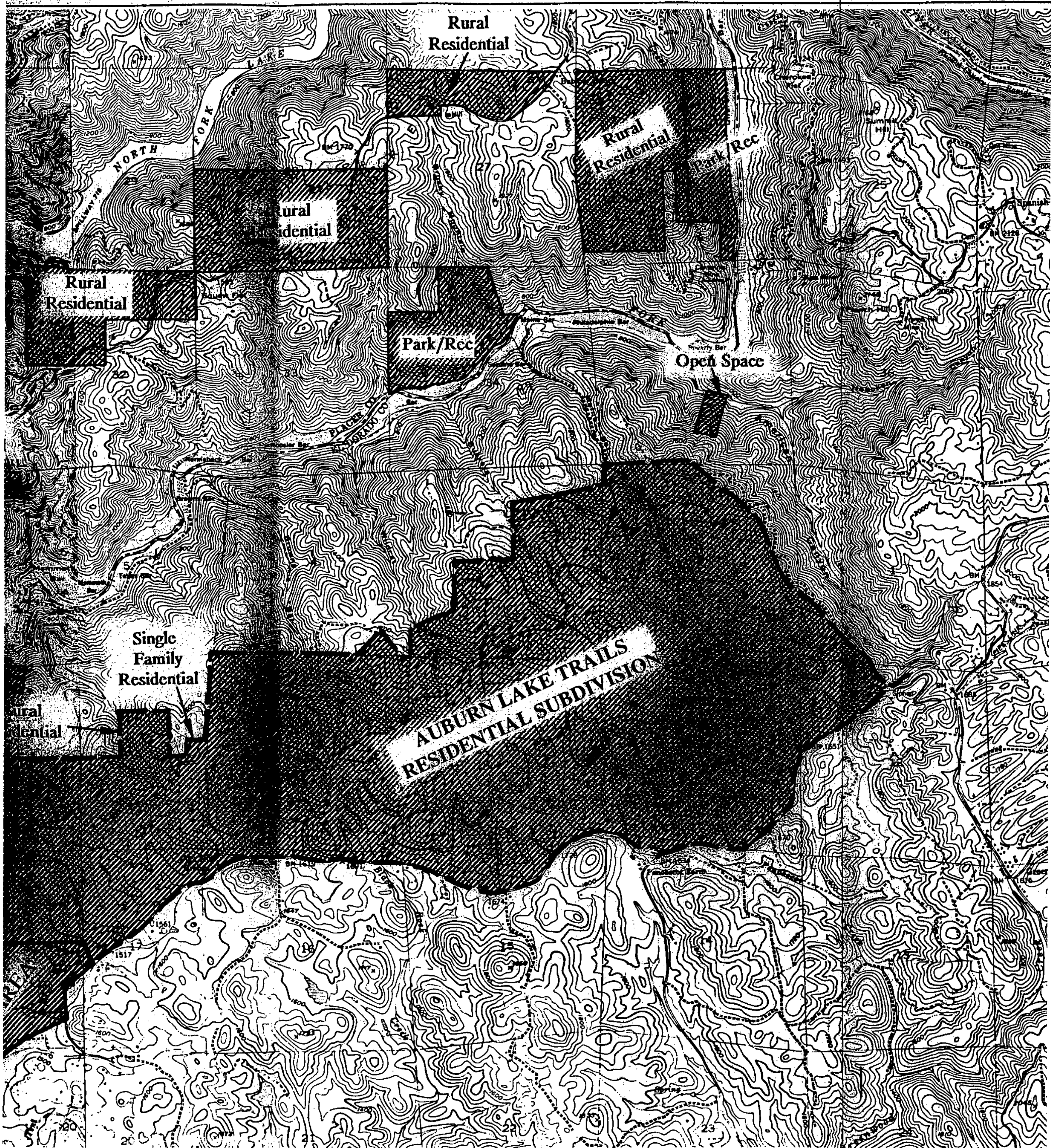
Cool Quarry Amphibolite. This site is located in El Dorado County approximately 7,000 feet north of the community of Cool and immediately west and downstream from the Cool Quarry (Figure 3-14). The site is a heavily vegetated ridge which drops steeply off into the Middle Fork Canyon. No development has taken place on this site and, other than the Cool Quarry to the east (refer to Figure 4.9-1), there is no development on adjacent properties. The site is visible from Highway 49 on the south and north. The site is situated approximately 100 to 500 feet lower in elevation than Cool. The residential community of Auburn Lake Trails is situated 3,000 feet to the southeast of the potential quarry. It is probable that the quarrying operation will be visible from some residences located on the ridgetops south and east of the quarry site.

The quarry site is designated "Open Space-Conservation" on the Cool-Pilot Hill Area Plan; zoning is Open Space. The site is on public land which is managed as part of the State Recreation Area. The land is under the jurisdiction of the Bureau of Reclamation.



Private Property in and Adjace

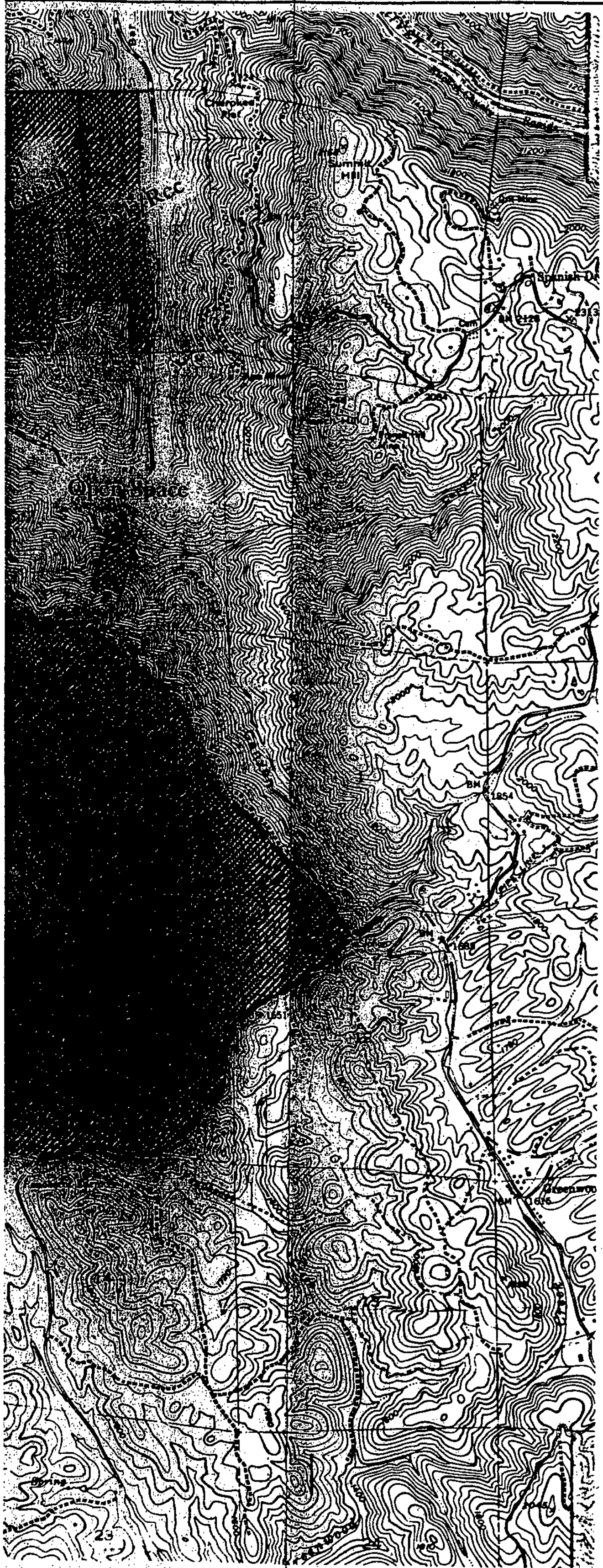
Source: USGS bas map; El Dorado County Cool-Pilot Hill Area Plan; Greenwood Area Plan and assessor's maps; Placer County General Plan and assessor's maps



Property in and Adjacent to the Auburn Project Segment

maps; Placer County General Plan and assessor's maps.

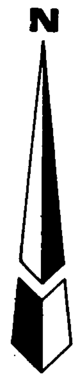
2



LEGEND

Private property in and adjacent to the Auburn project segment

Open Space — General Plan land use designation



Approx. Scale

1 Mile

t Segment

FIGURE 4.1-1

Adjacent land to the west is private land designated "Rural Residential" and "Open Space-Conservation"; zoning is RA-20, "Residential Agriculture," 20-acre minimum and "Mineral Resource" on the east.

Distant Aggregate Sources

Three of the aggregate sites under consideration are located some distance from the dam site vicinity. These are:

1) Bear River and Chevreaux Quarry in Placer County;
2) Mississippi Bar in Sacramento County; and
3) Yuba River Dredge Fields in Yuba County.

Mineral Land Classification

All three potential aggregate source sites are classified as MRZ-2 by the California Department of Conservation, Division of Mines and Geology. The Division of Mines and Geology (DMG) classifies land according to the presence or absence of significant Portland Cement Concrete (PCC) grade aggregate deposits. The land classification is presented in the form of Mineral Resource Zones (MRZ). The guidelines for establishing the MRZs are set forth in the DMG's Special Publication 51. All lands considered to be urbanized or urbanizing by the Office of Planning and Research, local lead agencies, or the DMG are assigned Mineral Resource Zone classifications (MRZ-1, MRZ-2, MRZ-3 or MRZ-4) based upon a geologic appraisal of the aggregate resource potential of the land. This appraisal includes study of pertinent geologic reports and maps, field investigation and sampling at outcrops and at active and inactive pits and quarries, and analysis of water-well logs and drill records.

Lands containing significant deposits of PCC-grade sand and gravel are classified as MRZ-2 and are evaluated to determine whether or not current uses of these lands preclude possible future mining. Areas currently permitted for mining and areas found to have land uses compatible with possible future mining are considered available for mining. These sectors are delineated and described in detail. Sectors are thus distinguished because the State Geologist judges that they meet the criteria for availability established by the State Mining and Geology Board (Special Report 156, Mineral Land Classification: Portland Cement Concrete-Grade Aggregate in the Sacramento-Fairfield Production-Consumption Region, 1988).

Bear River and Chevreaux Quarry

Land Management Plans. The Chevreaux Quarry is located in northern Placer County and is within the Meadow Vista General Plan area which was adopted in 1975. The quarry is designated Industrial on this plan. Adjacent and nearby properties are designated Agricultural/Residential and Farm. Minimum lot sizes in the vicinity of the quarry vary

from 10 acres to 100 acres. The Division of Mines and Geology classified the property owned by the Joe Chevreaux Company as MRZ-2 in 1983. This classification was requested due to the company's concerns with the continued growth of residential development on lands adjacent to their mineral properties. The MRZ-2 classification is applied in order to ensure that the mineral potential of the land is recognized in land use planning decisions.

At the upper end of Lake Combie, in the vicinity of the Chevreaux Quarry, the MRZ-2 zone extends onto the opposite shore. All of the lake itself is included in the MRZ-2 zone (see Figure 3-17) from the Nevada County General Plan and Zoning Ordinance. In Nevada County, properties immediately adjacent to Lake Combie are planned Rural Residential, 30-40 acre minimum parcel size. Zoning designations are A-1-19, General Agriculture, 10-acre minimum lot size; A-1-40, 40-acre minimum lot size; and P-ME, Public-Mineral extraction. The ME designation is provided for public awareness of the potential for mining to occur in selected areas. Residential development has taken place adjacent to the lake in Nevada County, approximately 2,000 feet southwest of the Chevreaux Sand and Gravel Plant.

Existing Land Uses. The Chevreaux Quarry is an existing quarry operation located in Placer County, approximately two miles north of the town of Meadow Vista on Combie Lake Road. The site is immediately adjacent to the Nevada County boundary. Residences are located adjacent to the south and east boundaries of the site, and a portion of Lake Combie is encompassed by the site on the west. The transport route is through a residential area and the town of Meadow Vista.

Mississippi Bar

Management Plan. Mississippi Bar is located on 160 acres of federally-administered land and is approximately one mile upstream from Nimbus Dam on the north side of Lake Natoma. It is approximately 1,000 feet from the shoreline of Lake Natoma and bounded on the north, west, and south by State of California lands, and on the east by federal lands administered by the Bureau of Reclamation. The site is presently leased to commercial suppliers of concrete aggregate. The General Plan for the Auburn Reservoir Project/Folsom Lake State Recreation Area (California Department of Parks and Recreation, 1980) suggests possible recreation facilities at Mississippi Bar that the DP&R could develop including dredging to create a new landscape/use area at the lake edge with shallow warm water lagoons, island, and channels for swimming, canoeing, sailing, exploring, and entrance station, picnic sites and a paved parking lot.

The site is classified MRZ-2; however, the Division of Mines and Geology recognizes parklands as having special status and mapped sectors are treated as a special class of sectors. Mississippi Bar is within a sector which is located within the American River Parkway. Special operating conditions have been applied to the aggregate extraction operations which restrict time of day of mining activities and restrictions on machinery and

hauling (USDI, BR, Mississippi Bar Aggregate Removal, Environmental Commitment Plan, August 1988).

Mississippi Bar Environmental Commitments

- Limit times of hauling aggregate; 8:30 am. to 3:30 p.m.
- Reduce dust emissions.
- Use existing topography to shield residences from noise.
- Restrict times of mining; 8:00 a.m. to 4:00 p.m.
- Keep noise from equipment at a minimum
- Shield mining activities from stables and equestrian activities.
- Warn riders about mining activities.
- Noise levels at Twin Lakes School will not exceed guidelines.

Existing Land Uses. The site is located below the river bluff and immediately south of an existing Orangevale residential area and Phoenix Park. No processing takes place on-site. The site is within the American River Parkway, and bike and equestrian trails travel through the bar and parts of the reclaimed aggregate mining boundaries. Excavations are within 200 feet of the bike path and within 300 to 400 feet of homes on Dredger Way. There is an adjacent commercial horse stable which rents horses to the public and sponsors trail rides on the parkway equestrian trail. The transport route for trucks hauling aggregate from the site is north on Main Avenue to Madison Avenue.

Yuba River Dredge Fields

Land Management. The Yuba River resource area, which is classified MRZ-2 by the California Division of Mines and Geology, extends along the Yuba River from upstream of the town of Smartville, downstream (southwestward) to the city of Marysville. Within the 10.3- square-mile area permitted for mining, there are nine PCC aggregate companies that are permitted to mine, of which two are inactive. The hydraulic wash deposits are a result of historic hydraulic mining which took place upstream on the Yuba River in the mid- to late 1800s. In the early 1900s, the aggregate was dredged for gold, and the dredge tailing area has been dredged and redredged intermittently to the present time (California Department of Conservation, Division of Mines and Geology, Mineral Land Classification: Portland Cement Concrete Grade Aggregate in the Yuba City-Marysville Production-Consumption Region, 1988).

According to Mike Bartlett of the Yuba County Planning Department, the dredge fields are designated "M-2," Industrial Extractive, on the Yuba County General Plan. Surrounding land uses are generally designated agricultural. The aggregate extraction operations are under use permit to the County and have reclamation plans on file with the County. The County is responsible for annual inspections of these operations under the requirements of the State Mining and Reclamation Act (SMARA) to determine compliance with the use permits and reclamation plans. Past inspections have revealed violations of permits by certain operators.

Existing and Planned Land Uses. The Yuba River Dredge Field is an existing, permitted, operation. Surrounding land uses are mainly agricultural including orchards adjacent to some of the active mining areas. The communities of East Linda and Hammonton are near the aggregate extraction areas, and there are scattered rural residences in the area. The County has received noise complaints from residents in the past. As a result, some operations may have restrictions on hours of operation (Bartlett, pers. comm., 1991).

4.1.2 IMPACT ANALYSIS

Consistency with Management Plans

According to the Draft Feasibility Report for the American River Watershed Study, all public lands within the project area will be retained. The USBR and CDP&R are expected to continue to manage existing lands for recreational uses. Temporary disruptions of recreational use will occur due to quarrying operations. Depending upon the selected quarrying site(s), recreational experiences may be altered from that which currently exists.

Middle Fork Sand and Gravel Deposits

Mining operations on the Middle Fork Gravel Bars would be within 10,000 feet of residential properties in El Dorado County (Auburn Lake Trails) and adjacent to properties designated for Open Space, Parks and Recreation in Placer County. Aggregate processing would take place north of Mammoth Bar, approximately 2,000 feet from the nearest property designated residential and 5,000 feet from existing residences in Auburn Lake Trails. Due to the depth of the canyon, it is probable that noise will not impact residential areas. Development on those properties designated Rural Residential on the Placer County General Plan, closer to the river in Section 26, may be subjected to visual impacts and noise (refer to Section 4.7, Noise, and 4.9, Visual Resources).

The mining operations would not be compatible with the recreation uses of the river. During the mining operations, the river would not be accessible for recreation; following completion of the mining, changes in the river flow characteristics caused by streambed alterations may have an effect on recreation in the river (refer to Section 4.10, In-stream Impacts). The private lands in Section 26 and 34 on the west and north sides of the Middle Fork are not developed and are designated Parks and Organized Recreation on the Placer

County General Plan. Due to the topography of these sites, their use is limited. However, both are immediately adjacent the gravel bars and any recreational use would be eliminated during the mining operations. Following the mining operations, characteristics of the adjacent river will be altered, significantly impacting any recreational use of these properties (refer to Section 4.8, Recreation).

<u>Old Cool Quarry (Spreckles)</u>

Because this quarry was in operation prior to enactment of Measure A, the requirement for maintaining the 10,000-foot buffer will not apply unless the quarry expands outside of its currently permitted area. Use of this quarry for construction of the Auburn Dam will necessitate an increase in the rate of mining, night lighting and longer operation hours. Blasting, vehicles, and aggregate crushing and screening equipment will produce noise and dust. Lighting will be required for nighttime operations. This increase in activity will significantly affect residences in the Auburn Lake Trails development which overlook the quarry (Class I). The nearest residences are approximately 2,000 feet away, across a ravine, from the quarry. Noise and lights will be an additional disturbance to these residents.

<u>Cool Quarry Amphibolite</u>

Due to proximity of the site to existing and planned residential areas, significant land use conflicts are anticipated to result from the development of this site as a quarry (Class I). It is projected that operations at the site would proceed on a 24-hour basis in order to meet project deadlines. Blasting, vehicles, and aggregate crushing and screening equipment will produce noise and dust. Lighting will be required for nighttime operations. Residences in the Auburn Lake Trails area would be subjected to noise, light and dust. While this site is not subject to Measure A, residences (including the community of Cool and Auburn Lake Trails) and land designated for residential uses are within the 10,000- foot buffer required of mining operations on private land within the county.

<u>Bear River and Chevreaux Quarry</u>

The primary impact associated with this quarry site are conflicts with nearby residential uses and impacts to land uses along the haul route. The volume of truck traffic necessary to transport the materials to the dam site would result in significant (Class I) impacts to residences, schools and businesses due to noise, dust and vibrations and traffic safety (refer to the Transportation, Noise, and Air Quality sections).

<u>Mississippi Bar</u>

An increase in mining activity may impact adjacent residential and recreational land uses. According to the U.S. Bureau of Reclamation, Environmental Assessment for the Mississippi Bar Aggregate Removal (April 1988), the stables and equestrian trail would be impacted by the mining operation. The bike path is in a reclaimed area; therefore, it would not be

affected by the mining activities. However, users may be subjected to high noise levels. Access to the bike trail is located on Main Avenue. Increased truck traffic may create a safety hazard for bicyclists near Dredger Way.

Transport of the aggregate material from this site would result in significant disturbance to nearby residential areas which would be subjected to increased noise and vibration, dust and air pollutant emissions from trucks (refer to Noise and Recreation sections). These are considered significant unavoidable impacts (Class I).

Yuba River Dredge Fields

An increase in the annual amount of material extracted from the sites as allowed under the use permits and reclamation plans would require amendments to the reclamation plans.

While there has been no discussion or concern regarding possible impacts to adjacent agricultural operations, an increase in activity may result in increased dust emissions from the extraction sites which could affect crops. Current dust abatement measures may be adequate. These impacts are considered significant, but mitigable (Class II).

No substantial residential or urban development is nearby the sites; however, there are scattered rural residences. The County has received noise complaints in the past. Expanded hours of operations could result in noise impacting some residents in the area. These impacts are considered significant but mitigable (Class II).

4.1.3 MITIGATION MEASURES

Middle Fork Sand and Gravel Deposits

No access to the river will be permitted during mining operations. Therefore, significant unavoidable impacts would occur to recreational use of the river during mining operations. There is no mitigation that would reduce this impact.

Old Cool Quarry and Cool Quarry Amphibolite

- Use residential grade mufflers and engine enclosures on machinery.
- Restrict hours of operation to 7 a.m. to 7 p.m., Monday through Friday.
- Use dust abatement measures; these measures will reduce but not eliminate significant impacts.
- No night blasting between 7 p.m. and 7 a.m.

- Locate processing operations at bottom of slope near river in order to provide topographic visual and noise screening.

Those measures which restrict hours of operations are not compatible with the project time constraints; therefore, significant unavoidable impacts would occur with respect to land use compatibility.

<u>Bear River and Chevreaux Quarry</u>

Impacts identified are significant and unavoidable. Measures restricting the hours of operation and the number of truck trips would mitigate impacts to some degree; however, these restrictions would not be compatible with project time constraints.

<u>Mississippi Bar</u>

Measures identified in the Environmental Commitment Plan provide mitigation; however, these measures would place limitations on the excavation and transport operations which are incompatible with the project time constraints. Therefore, impacts are significant and unavoidable.

<u>Yuba River Dredge Fields</u>

Impacts associated with aggregate extraction at this site can be mitigated to less than significant levels.

- Use dredge tailing where feasible to create topographic barriers which will reduce noise from processing plants.

- Use dust abatement procedures in mining and processing operations to reduce dust emissions from the site.

- Compliance with existing permits and mining and reclamation plans.

4.2 PUBLIC HEALTH/SAFETY

Mining operations can result in several potential impacts to public health and safety. Public health and safety impacts associated with mining operations typically fall into two categories (1) impacts related to mining processes and plant operations such as: accidents involving misuse of toxic materials, mining equipment, explosives and open pits; potential water contamination (caused by misuse or improper construction of settling ponds), wildfires, air pollutants from mineral particles and equipment emissions; increased risk of vehicle accidents from increased transportation; and (2) attractive nuisances, meaning excavation areas are often attractive to certain individuals as a place to play.

The following discussion of public health and safety concerns associated with aggregate mining focuses on the Middle Fork sand and gravel deposits, proposed quarry operations and distant sand and gravel operations.

4.2.1 EXISTING CONDITIONS

Middle Fork Sand and Gravel Deposits

In the project area, the Middle Fork of the American River is contained within as a steep canyon characterized by a variety of vegetative cover types such as chaparral, riparian scrub shrubs and annual grasses. Access from Highway 49 and Foresthill Road to the river is limited with some access points requiring 4-wheel drive vehicles. The area is a designated State Recreation Area which supports a diversity of recreational activities for thousands of people each year.

Old Cool Quarry (Spreckles)

The Old Cool Quarry is situated on steep terrain on the south side of the Middle Fork. It is adjacent to Highway 49, which is the main transportation route out of the area. The quarry has been terraced in order to stage mining operations away from the highway. The surrounding land is densely vegetated with chaparral, various oak and pine species. Several homes in the Auburn Lake Trails subdivision overlook the quarry. Some houses are situated along a ridge and have direct view of the quarry. Noise generated from the Cool quarry can be heard on those lots located close to the canyon ridge. Landforms separate most of the subdivision from operational impacts, however.

Cool Quarry Amphibolite

The proposed amphibolite quarry is situated adjacent and directly to the west of Old Cool Quarry. Old Quarry Road, which is a portion of the Western States Trail, is located at the base of the site just above the river. The terrain and vegetation of the site are typical of

those of the Middle Fork canyon. There are no residential areas within close proximity to the potential quarry site.

Bear River and Chevreaux Quarry

The Chevreaux Quarry is located in northern Placer County approximately 2 miles north of Meadow Vista on Combie Lake. Residences are located adjacent and to the scuth and east of the site. The northern portion of Lake Combie is within the site boundary with the majority of the lake lying to the south. Access to the Chevreaux property is provided from Interstate 80 via Placer Hills and Lake Combie Roads, which are two-lane rural roads passing through the central business district with existing school and commercial development.

Yuba River Dredge Fields

The Yuba River resources area extends along the Yuba River from upstream of the town of Smartville, southwest to the town of Marysville. Small communities such as East Linda and Hammonton are within three to four miles of the aggregate extraction areas, and there are rural residences scattered in the immediate area. Surrounding areas are primarily in agricultural production. Aggregate trucks commonly travel southwest on Hammonton-Smartville Road to State Highway 65.

Mississippi Bar

The Mississippi Bar site contains an existing mining operation located on 160 acres of federally-administered land. The site is approximately one mile upstream from Nimbus Dam adjacent to Lake Natoma. Residential areas are within close proximity to the operation, and the American River Parkway bike trail transects the reclaimed area of the site. Existing transport routes traverse residential areas of Orangevale prior to accessing Highway 50.

4.2.2 IMPACT ANALYSIS

Impacts to public health and safety associated with sand and gravel or quarry operations are very similar. However, certain elements in proposed alternatives are unique to the site, the method of extraction, or the method of transport to the dam site. The following discussion of public health and safety impacts identifies unique impacts associated with each site. Common impacts associated with all proposed sites follows.

Middle Fork Sand and Gravel Deposits

Mining operations on the Middle Fork would cease during the flood season, or during periods of intense rains. In the event of flooding, mobile equipment would need to be removed from the flood area. Immovable equipment such as the conveyor system and

processing plant would be located above the floodplain and would remain in place for the entire year. The volume of truck traffic necessary to transport heavy mining equipment would result in a short-term significant impact to existing traffic. The Middle Fork supports many recreational activities and is used by several thousand people every year. Many people travel from distant locations, such as the Bay Area, to enjoy several recreational opportunities. The presence of mining operations within an area with considerable historic public access, would create potential safety problems with persons climbing or riding the conveyor system or mining equipment. This would be a significant adverse impact (Class II) which could be mitigated.

<u>Old Cool Quarry (Spreckles)</u>

As with the Middle Fork sand and gravel deposits, this alternative would have safety problems associated with persons climbing or riding on the conveyor system; however, the conveyor system would be approximately seven miles in length as opposed to twelve miles for the Middle Fork sand and gravel alternative. Considerable amounts of heavy equipment activity would be necessary during construction and dismantling of the conveyor system. These impacts to public safety are considered short-term but significant (Class II impact) and would require mitigation.

<u>Cool Quarry Amphibolite</u>

Removal of overburden at the proposed site would require excavation of a large amount of trees, shrubs, topsoil and boulders. Improper stockpiling of the overburden could result in a safety hazard. This would be a significant adverse impact to public safety (Class II). Limiting access to the area via signage and fencing would mitigate this impact; however, there is still the risk that some people would attempt to enter the area. In addition, this alternative would have the same safety problems concerning the conveyor system as described in the previous alternative sites.

<u>Bear River and Chevreaux Quarry</u>

Transportation routes from the Chevreaux property would be via rural two-lane roads that are the major link between the town of Meadow Vista and Interstate 80. According to the Corp's Special Aggregate Report, 700 to 900 deliveries would occur each day. This would be an unavoidable significant adverse impact (Class I) to public safety which could not be mitigated.

<u>Yuba River Dredge Fields</u>

Based on a review of potential haul routes, the most likely route to the dam site is approximately 40 miles. This potential truck route would be via Hammonton-Smartville Road through Linda to Highway 65 south via North Beale Road. At the town of Lincoln, the route would connect to State Route 193 which connects to Interstate 80 just north of

Indian Hill Road. These routes are primarily 2-lane rural roads. This would require trucks to transit through the commercial districts in Linda, Wheatland, Lincoln, and Auburn. Much of this route has adjacent residential uses. A delivery rate of 40 trucks per hour for an approximate period of two years would result in a significant adverse impact (Class II) to public safety in all these towns. The alternative transport by rail would mitigate the impact to public safety to a less-than-significant level. A rail spur would be linked with the Southern Pacific rail line. This alternative is currently being negotiated.

Mississippi Bar

The transportation routes would most likely proceed via Main Avenue which is a collector street that travels through residential neighborhoods to Hazel Avenue, a major 4-lane arterial which runs north through Orangevale and south to U.S 50. Madison Avenue, a 4 to 6-lane arterial, is currently used as a direct route to Interstate 80. A delivery rate of 40 trucks per hour would mean a truck traffic increase of 300 percent. This would result in an unavoidable significant short-term impact (Class I) to public safety within these residential areas and upon local roads.

All Proposed Sites

All the proposed sites have common operational risks such as the presence of explosives, fuel storage, equipment malfunctions and public access which may result in a significant adverse impacts (Class II) to public safety; however, effective mitigation is available. Quarry operations would utilize explosives. These problems are discussed below.

Storage of equipment fuels, petroleum products and explosives on site would be as necessary. This increases the risk of spills, explosions and fire.

Releases of waste petroleum products could result from improper storage and handling of waste oil, lubricating fluids and other materials. Potential water contamination would exist if such releases occurred on permeable ground or near active streams. This would constitute a potentially significant impact.

Aggregate operation would increase the possibility of wildfire in the surrounding area. All the alternative sites are located in the foothills and are surrounded by areas with concentrated amounts of native vegetation and annual grasses. An exception may be the Yuba River dredge fields. Summer months are hot creating very dry conditions and increasing the chances of wildfires. This presents a potentially significant impact on public safety.

Worker injury arising from equipment malfunction or unsafe practices could be considered a potentially significant impact. Malpractice of workers associated with mining operations could put public health and safety at risk.

Public access is relatively easy on all proposed sites and could put public health and safety at risk.

4.2.3 MITIGATION MEASURES

Significant impacts caused by transporting source material by truck from the alternative sites to the proposed dam site would be unavoidable and unmitigable. Other significant impacts, which for the most part are common to all sand and gravel or quarry operations, would be mitigable through implementation of appropriate operational controls and restrictions to access.

Middle Fork Sand and Gravel Deposits

- Provide an evacuation plan focusing on annual evacuation during the flood season and emergency evacuation of mobile equipment from the Middle Fork.

- Require public access closure of the Middle Fork during mining operations on the river. Post signs along Highway 49, Foresthill Road, and other areas providing access to the Middle Fork indicating closure of the Middle Fork segment during mining operation.

- Require fencing around processing operations to prohibit public access.

- Organize and implement a public information program in order to discourage public use of the Middle Fork canyon and river for recreational purposes throughout the duration of the mining operations.

- Fuel tanks and their attendant containment structures must be constructed to conform to applicable State and County requirements to prevent spillage and/or other discharge to ground and surface waters. Typical facilities would be constructed to contain the contents of the tank itself together with precipitation resultant of a 100-year flood event. Maximum security measures for fuels and explosives would be incorporated into standard practices of the mining operations.

Cool Quarry Amphibolite

- Fencing around overburden areas and posting warning signs.

Transportation Impacts Common to All Sites

- Impacts associated with truck transportation of source material are unmitigable. Damage to roads, which inflicts risk to public safety itself, shall be repaired to appropriate road standards.

All Sites

- Arrange for the collection and proper disposal of waste oils and materials.
- Require a timely submittal of business plans in accordance with the requirements of Chapter 6.95 of the State Health and Safety Code.
- Require routine vehicle and equipment maintenance in a designated shop area with an impermeable surface.
- Submit to periodic fire safety inspections by the local jurisdictions's Fire District.
- Organize an on-site fire fighting unit to provide a primary response to fires.
- Provide for necessary fire fighting equipment such as Purple K foam to extinguish on site petroleum fires, and work with the local Fire Department to establish fire protection equipment specifications.
- Comply with Mining Safety and Health Agency (MSHA), and CAL-OSHA worker safety requirements; post safety rules in conspicuous locations.
- Conduct safety meetings in accordance with MSHA and CAL-OSHA regulations.
- Provide worker training in safe operation of equipment.
- Formally designate site safety officer to identify and eliminate unsafe equipment and practices.

4.3 WATER QUALITY

The water quality analysis considers the potential impacts associated with sand and gravel mining and aggregate processing at the alternative sites. A primary concern for source locations near surface waters is the potential for increased erosion and sedimentation of receiving waters. Increased turbidity levels can adversely affect water quality and the aquatic habitat.

The aggregate processing facility, regardless of the site selected, would use water in the washing and processing operations. The disposal of the wash water would be subject to waste discharge requirements to be established by the Central Valley Regional Water Quality Control Board (RWQCB).

Potential impacts to water quality are described in qualitative terms for each of the alternative aggregate material source locations. Mitigation measures are discussed at the end of this section. Successful implementation of the mitigation measures would significantly reduce the potential for water quality degradation. A water quality monitoring program may also be required by the RWQCB for sites near surface water drainage and for settling/infiltration ponds. General requirements of such a program are also discussed in the mitigation measures section.

4.3.1 EXISTING CONDITIONS

Middle Fork Sand and Gravel Deposits

The Middle Fork is one of three major forks within the 2,631-square-mile drainage basin of the American River. The American River includes natural areas and those that have been modified by human activity to meet recreational and water-supply needs. The Middle Fork drainage basin is approximately 616 square miles (see Figure 4.10-2).

Beneficial Uses and Water Quality Objectives. Water quality management by the Regional Water Quality Control Board includes establishment of beneficial uses and water quality objectives. Protection and enhancement goals of the identified beneficial uses determine the overall water quality objectives. The beneficial uses of the American River include:

- Municipal and domestic supply
- Irrigation
- Stock watering
- Water contact recreation
- Canoeing and rafting
- Non-contact water recreation
- Hydroelectric power generation
- Warm freshwater habitat
- Cold freshwater habitat

Spawning (warm water)
Spawning (cold water)
Migration
Wildlife habitat
Riparian habitat

The primary beneficial uses in the vicinity of the Middle Fork bars considered in this analysis include domestic water supply, contact and non-contact recreation, cold water spawning, cold freshwater habitat, and wildlife habitat.

Water quality objectives are developed to meet State and Federal requirements for maintenance of water quality. For the American River, non-degradation is the operational policy of the Regional Board. The non-degradation policy calls for the protection and maintenance of high-quality water resources at background levels of quality, which means that pollutant concentrations in the American River must not increase to the extent that beneficial uses are affected. The following water quality objectives for the American River are part of the non-degradation policy:

- No increase beyond natural background levels for turbidity;
- No bottom deposits other than natural causes;
- No floatables, oil, and grease, other than natural causes;
- No significant change in normal ambient pH value; pH shall not be depressed below 6.5 or raised above 8.5 as a result of waste discharges;
- No substance will be added which produces aquatic growths in the receiving waters to the extent that such growths cause nuisance or damage to any of the beneficial water uses;
- Bacteria levels will be those recommended by the California State Department of Public Health;
- Water shall remain free from adverse temperature changes resulting from waste discharge or other activities of man;
- No substance which produces deleterious effects upon beneficial uses shall be discharged to receiving waters; and
- No significant increase in color beyond natural background levels.

Overall, the water quality of the American River is considered good for the designated beneficial uses. Recreational overuse, improper land use and poorly managed mining operations are considered potential sources of water quality problems for the upper American River basin. Mining operations have a history of various water-quality problems in the upper basin. A major concern associated with any mining activity is increased sedimentation. Incidents of increased sedimentation from mining activities near the river have resulted in significant impacts on aquatic organisms near and downstream from the activity. Contamination of the water by trace mineral seepage from mine spoils or stockpiles is also of concern.

Old Cool Quarry (Spreckles)

The Old Cool Quarry is located in the American River Middle Fork Canyon, approximately 5 miles upstream of the proposed dam site. Figures 3-15 and 3-16 depict the quarry site. The quarry is currently operating with a processing capability of 600 tons per hour.

The site is approximately 500 yards south of the river channel. As discussed previously, the RWQCB has instituted a number of beneficial uses and water quality objectives aimed at protecting and enhancing this high-quality resource.

Wash water is obtained from on-site wells and discharged on-site into settling ponds that have been developed from areas of previous excavation. Two ponds are presently being used. The primary site, the "south pit" is located in the southern area of the quarry, roughly 1/2 mile distant from the river channel. The northern pond is near the northern boundary and lies roughly 500 yards south of the river channel. Water use and disposal within the operation is contained within the quarry site (Bartley, pers. comm., 1991). Surface drainage is also contained on-site.

Cool Quarry Amphibolite

The source identified as Cool Quarry Amphibolite is located immediately west of the Old Cool Quarry (see Figure 3-1). This site is presently undeveloped and would require extensive site preparation prior to the actual mining of aggregate materials.

The site is within the Middle Fork drainage area, roughly 500 yards south of the river channel. No significant surface drainages have been identified.

Bear River and Chevreaux Quarry

The Bear River and Chevreaux Quarry are located near the town of Meadow Vista, approximately 11 miles north of the dam site (see Figure 3-1). Current alluvial mining operations of the Bear River-Lake Combie deposits extend upstream from Lake Combie Dam for over two miles. Quarrying and crushing operations are located near the river.

The RWQCB has established beneficial use categories and water quality objectives for the Bear River, similar to those discussed for the American River. According to the RWQCB, beneficial uses of the Bear River include the following:

- Municipal and agricultural supply
- Recreation
- Aesthetic enjoyment
- Preservation and enhancement of fish and wildlife and other aquatic resources

The RWQCB is responsible for protection of these uses and may set discharge limits on aggregate operations. As discussed for the Middle Fork sand and gravel deposits, a number of discharge limits and non-degradation policy requirements may be applicable to future operations.

Lake Combie is managed by the Nevada County Irrigation District. The district manages the lake's water supply for agricultural irrigation. Nearby residents also use the lake for boating and other water recreation. There are no public water/recreation facilities at the lake.

Mississippi Bar

Mississippi Bar is located on the lower reach of the American River, below Folsom Dam (see Figure 3-1). The Mississippi Bar operation is located within 1,400 feet of the Lower American River. This stretch of the river has high value for habitat and recreation uses. Water quality is also important as the river is used as a source of drinking water (see discussion of beneficial uses under Middle Fork section).

Presently, reclamation of mined areas includes development and enhancement of recreation and habitat areas. These efforts are part of an Environmental Commitment agreed upon by the operators and USBR. Specific commitments were identified by an environmental assessment conducted prior to an expansion of aggregate removal operations in 1988. The objective for water quality is:

> *Prevent increased sedimentation and erosion created by or resulting from the project from entering the river.*

The approach to minimizing erosion and thereby protecting the river water quality is as follows:

> *Prior to beginning mining, the successful bidder will develop plans for repairs, channels, and checks to manage runoff and minimize erosion and turbidity. Reclamation will approve these plans and monitor their implementation. After excavation, graded areas will be seeded...*

Implementation and monitoring of these measures has prevented significant degradation of water quality in the vicinity of the Mississippi Bar. Recreation and habitat uses are still viable uses in the area.

Yuba River Dredge Fields

The Yuba River aggregate reserves are located near the town of Marysville (see Figure 3-1). The deposits along the Yuba River are similar to those found along the American River (Middle Fork Bars and Mississippi Bar). The area is actively mined by three companies. These operations are located adjacent to the river channel. At the Western Aggregate plant, a series of settling ponds are designed to capture sediment before water is discharged into the river. Water quality of the river and settling ponds is monitored on a regular basis. In the past, high silica content was found in the river water; however, this problem has not been detected recently (Clausen, pers. comm., 1991).

4.3.2 IMPACT ANALYSIS

Middle Fork Sand and Gravel Deposits

The mining activities proposed along the Middle Fork would potentially affect the American River water quality within the stretch of river proposed for excavation and downstream of the operations. A site map of the ten selected bars is provided on Figure 3-2. Figures 3-3 through 3-14 present aerial photographs of the sand and gravel deposits. As can be seen from the aerial photographs, the deposits lie adjacent to and within the Middle Fork river channel. The preliminary aggregate study prepared by the COE indicates that this alternative would involve removal of the entire sand and gravel deposit existing within each of the identified bars.

Impacts to water quality are significant if the operation results in an increase above ambient levels of pollutants within the Middle Fork. Water quality criteria include the California Regional Water Quality Control Board's general policy of non-degradation for all receiving waters. Other relevant impact evaluation criteria exist in Federal standards (Water Quality Act, 1987), primarily for point sources of pollution, but also for ambient levels of pollutants in surface waters.

Perhaps the biggest water quality threat posed by mining of the Middle Fork bars is elevated levels of turbidity and associated increases in siltation. In addition to site preparation and general activity adjacent to the river channel, the excavation operation and the washing process have the greatest potential to act as sediment sources. Siltation of gravel beds smothers aquatic eggs, reduces benthic (river bottom) populations and reduces protective cover for larvae. Uncontrolled turbid discharges into the Middle Fork would reduce light penetration into the water and diminish the productivity of benthic and planktonic communities within the river and Folsom Reservoir (see Section 4.5). In general, significant

water quality impacts resulting from aggregate operations near waterways can be avoided by maintaining a separation between the operation and the waterway.

Because the sand and gravel deposits are located within the 100-year floodplain, impacts from high water flows would also be potentially significant. The primary impacts anticipated on surface water quality include sedimentation and erosion from operation of heavy equipment along bank slopes, potential resuspension of river sediments caused by dredging and heavy equipment operations near the river, and erosion of exposed sites during storms. Other potential impacts include uncontrolled discharges of wash water and spillage of petroleum products.

Due to the extent of the proposed mining activity and the close proximity of the operation to the river channel, it is expected that significant short-term unavoidable water quality impacts would result from full implementation of this alternative. These impacts can be reduced through implementation of appropriate mitigation measures (see Section 4.3.3), but not completely eliminated (Class II). Some residual impact will necessarily be realized.

Site Preparation. As assumed in the project description, excavation would begin at Mammoth Bar and proceed upstream to Cherokee Bar. It is possible that two bars may be mined simultaneously. Preparation of each bar area would involve access route construction and extension of the conveyor system. Vegetation removal and soil disturbance caused by these operations could accelerate the erosion of top soils. Loose soils may be transported by wind or overland flows to the river channel potentially increasing the sediment load in the vicinity of construction activity. These impacts are considered potentially significant (Class II). The preparation of each site would be a short-term function. Several measures can be implemented to reduce the potential for these impacts as discussed in the mitigation section.

Excavation/Mining. The excavation operation would involve placement of a dragline bucket (see Figure 3-23) on top of an individual bar. Excavation would likely begin near the river and proceed toward the floodplain margin. The bucket would repeatedly scoop sand and gravel from the bar down to a depth near bedrock. If allowed to enter the river channel, releases of fine-grained material disturbed during the excavation process has the potential to degrade water quality in the Middle Fork by increasing suspended sediment levels. During the operation, berming of the "live" channel would serve to isolate excavation activities and prevent turbid discharges to the river.

Once excavation is complete, the mined area adjacent to the river channel poses a water quality threat. The disturbance and reconfiguration of the river banks caused by removal of gravel from within the river's floodplain would initially result in increased erosion rates during the initial seasonal flows. This process is facilitated by removal of the outer armoring layer of coarse-grained material which currently covers the surface of the individual bars. Because the operation is likely to occur over a period of several years, initial high flows for several years can be expected to contain relatively high levels of fine-

grained sediment. The term "relatively" is used because seasonal high stage events naturally contain elevated sediment loads.

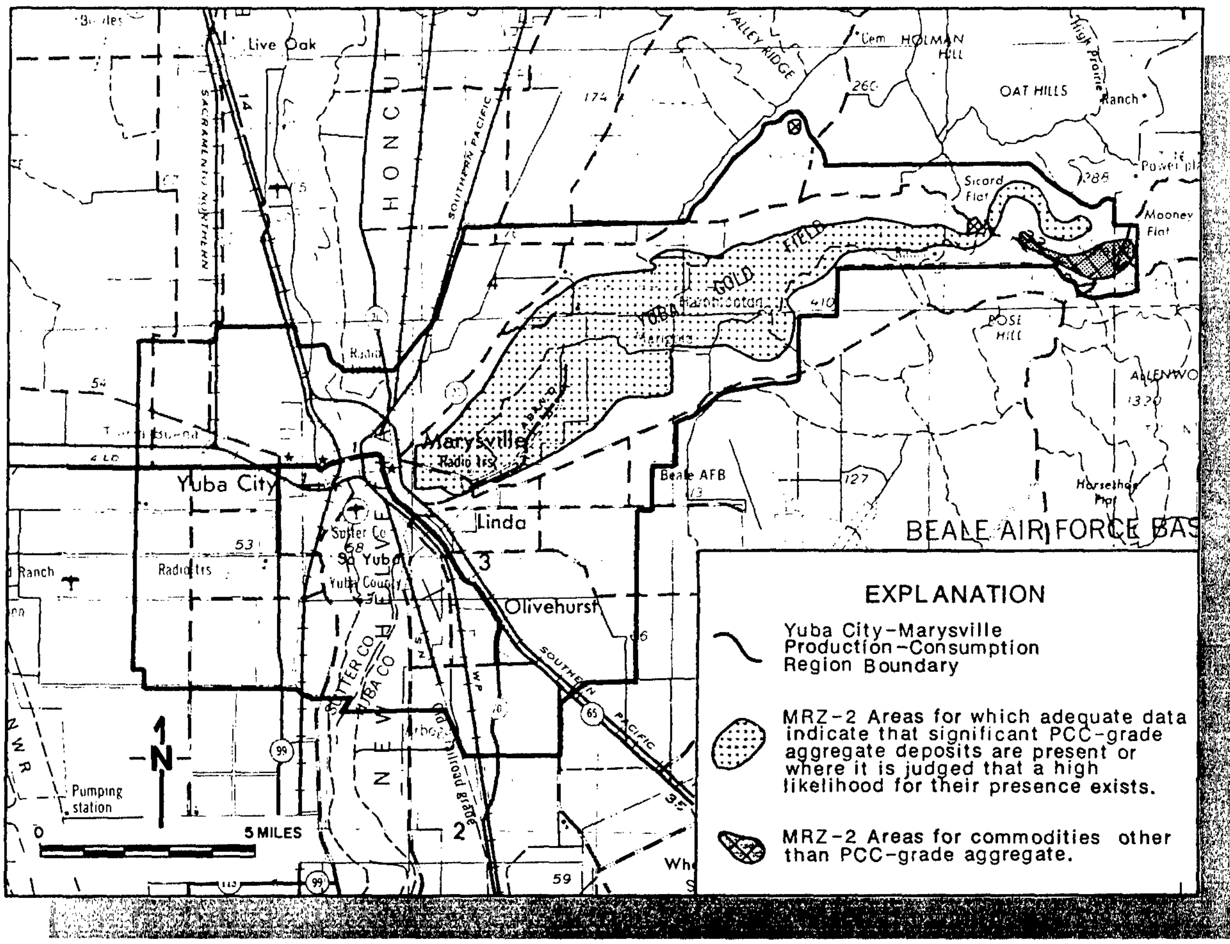
This impact is considered significant but mitigable (Class II). Some residual impact will be realized. However, it is difficult to speculate on the magnitude of the residual water quality impact without further investigation.

Surface Runoff/Erosion. Excavation of the bars would necessitate removal of the coarse-grained surficial armor layer as well as any vegetative cover on the bars. Both the armor layer and the vegetation serve to limit natural rates of erosion. Removal of these protective devices would locally accelerate the natural erosion process and increase the sediment contribution from the mined reach of the river until these natural protective devices become reestablished.

The proposal suggests that mining activities would occur 8 to 10 months of the year, excluding the rainy season. Areas mined during the dry months would be exposed and vulnerable to erosion from runoff generated on adjacent canyon escarpments. The vulnerability could be short-lived; several high stage events would flush most of the easily erodible material downstream (see excavation impacts above). In the absence of such an event or series of events, an incremental increase in sediment output would be realized until the river had regained a state of equilibrium. These types of impacts are significant but mitigable if effective erosion control measures are implemented (Class II).

Processing Plant. Processing of the pitrun material will result in the generation of waste fines or materials too small for use in concrete manufacture. These silts and clays would be washed out of the aggregate processing by water sprays on the "wet side" of the plant. Washing can produce water containing dissolved salts, metals and sediment. The same contaminants can be added to the water used to transport the materials during various processing steps. In some processes, large amounts of water are used and discharged, however, much of the water can be recycled and little if any may be discharged. Excess water and the fine materials generated at the aggregate plant would be collected in a pipe and transported to settling/infiltration ponds. Settling ponds are one of the most common methods used to control suspended sediment in waste water. Such ponds function by holding water long enough for much of the sediment to settle and hence must be designed with respect to the predicted frequency and volume of discharge.

A series of lined ponds would be developed to allow for settling of fines from process wash water. The solid material in the flow would settle to the bottom of the ponds before being reused in the washing process or discharged to the river. Periodic dredging and removal of these fines from the ponds would be necessary for proper operation of the system. The proposal suggests locating the ponds downstream of the mining sites and above the dam site. Because the amount of space available for such ponds is limited, flocculating agents may be used to enhance the settling process.



Yuba River Mineral Resource Zone

Source: Calif. Division of Mines and Geology, 1988

FIGURE 3-19

The settling of suspended sediment can be promoted by adding flocculates such as ferric compounds, lime, aluminum sulfate and various polymers. These reagents are ordinarily used after larger particles have been removed. Their effectiveness varies with the specific characteristics of wastes to be treated. Additional precautions may be necessary to ensure that these waters do not overflow into the river channel.

Regulatory Requirements. The RWQCB is responsible for protecting the beneficial uses of the state's water resources. If the RWQCB's discharge standards are exceeded, the beneficial uses of the American River could be jeopardized. To prevent such occurrences, the Regional Board issues waste discharge permits to aggregate producers located on rivers or streams to regulate waste water disposal. These permits generally prohibit the following:

- The direct discharge of wastes to surface waters or surface water drainage courses;

- The discharge of solids, including soil, silt, clay, sand, and other organic and earthen materials to surface waters or surface water drainage courses.

Aggregate producers are required to abide by these restrictions by discharging their waste water into settling ponds, where suspended solids settle out and are removed. In addition, waste discharge permits usually have the following specifications:

- Neither the treatment or the discharge shall cause a pollution or nuisance as defined by the California Water code, Section 13050;

- The discharge shall not cause degradation of any water supply;

- The discharge shall remain within the designated disposal area at all times;

- Collected screening, sludges, and other solids removed from liquid wastes shall be disposed of in a manner approved by the Executive Officer;

- Reclaimed wastewater shall meet the criteria contained in Title 22, Division 4, California Administrative Code (Section 60301, et seq.); and

- The dissolved oxygen content of holding ponds shall not be less than 1.0 mg/l for 16 hours in any 24-hour period.

According to the area engineer, the RWQCB would require establishment of a monitoring program that includes samples from the American River and the settling ponds (Batkin,

pers. comm., 1991). A monitoring program would need to be determined and implemented prior to any site preparation activities in order to establish baseline water quality data. The RWQCB would require on-going monitoring throughout the mining process. Typically they require monthly monitoring of the receiving water (Middle Fork) and daily observation of the settling ponds. The information obtained through the monitoring program must be written up and a report submitted to the board on a semi-annual basis. The monitoring and reporting conditions would become part of the Waste Discharge Requirement order imposed by the RWQCB.

Old Cool Quarry (Spreckles)

This alternative would involve expansion of the existing Old Cool Quarry operation from the current production rate of 300 tons per hour to 1,000 tons per hour; more than tripling current production levels and water usage. The relatively long history of quarrying at the site has created several large surface excavations as well as a network of underground workings. Figure 3-15 is an aerial photograph showing the operation. The bottom of some of the quarry pits intercept the local water table; ponded water was evident during a site visit in July. Standing water was also observed in the underground workings.

Potential adverse water quality impacts of this alternative are potentially significant (Class II) but mitigable using measures already in place. The current operation obtains and discharges process wash water on-site. On-site groundwater resources are used to supply wash water. Used wash water is discharged into one of the unlined excavation pits which serve as settling ponds. The fact that the existing processing operation and settling pits are located away from any waterways (see Figure 3-15) preclude the possibility of turbid discharges to sensitive watercourses.

Stormwater runoff is directed to one of the on-site pits or is allowed to drain by sheetflow into the surrounding hillside vegetation. Most of the material exposed by the quarry operation consists of unweathered rock which is highly resistant to erosion. Overburden is deposited on-site in abandoned quarry excavations. This eliminates potential impacts related to erosion of this material.

The above discussion assumes that an expanded operation would operate in a manner similar to the existing quarry. The RWQCB would review any expansion in operating plans to determine a need for Waste Discharge Requirements. However, on the basis of the analysis presented above, it is concluded that potentially significant water quality impacts related to utilization of the Old Cool Quarry could be mitigated to a less than significant level.

Cool Quarry Amphibolite

Development of a new quarry would be necessary if this alternative source is selected.

Site Preparation. The first stage would be removal of overburden materials. The terrain of the site is steep and heavily vegetated. Removal of overburden material and vegetation would increase the potential for soil erosion. Increased turbidity levels are commonly observed as a result of stormwater and snowmelt discharges to waterways. The contribution from the new quarry site would depend on the amount of area left exposed. Significant impacts to water quality could occur if the rainy season hits while the disturbed surface is exposed (Class II). The loosened material would be susceptible to entrainment in overland flows. Due to the distance to the river channel (roughly 800 vertical feet), much of this material may drop out and settle prior to reaching the river. Various stormwater diversion techniques such as berms or dikes which channel surface flows in a particular direction may be employed at the quarry site. The use of such erosion control features could significantly reduce the potential for pollution of the river.

Excavation/Mining. Mining of the Cool Quarry Amphibolite would be similar to other quarry operations, such as the Old Cool Quarry. Disturbance of the site would increase the potential for water erosion during storms. Stockpiles would also be a potential source of erosion. Due to the steep topography at the site, careful planning of the excavation would be necessary to minimize the amount of erosion. Control of surface drainages could effectively prevent significant erosion and subsequent pollution of the American River. Measures must also be implemented to prevent spills or seeps of fuel or other petroleum products into the local waterways. Impacts related to this aspect of the operation are considered significant but mitigable (Class II).

Processing Plant. The processing operations at this site would be similar to the other alternatives. The "wet plant" side would generate wastewater and fines requiring disposal. Initially, space for settling/infiltration ponds may be limited. Once a significant quantity of material has been removed, old cuts may be suitable for settling ponds similar to Old Cool Quarry. The design of the ponds must consider the site topography, mining sequence and distance to surface water and to groundwater. Additional site-specific investigations would be necessary to better evaluate this site.

Bear River and Chevreaux Quarry

The operations at Bear River-Lake Combie include dredging within the lake and along the river shores. Although the area is within a designated mineral resource zone, care must be taken to prevent significant deterioration of water quality. The Chevreaux operation processes rock for aggregate through a series of crushers, screens and classifiers. According to the operator, no waste material is generated by the process.

If this site were selected as the aggregate source for the Auburn Dam project, the increased level of production would create wastewater and materials. Disposal of these wastes would be as discussed for other source alternatives. Consistent with current practices at the site, water quality degradation would be avoided through the use of settling ponds. Additional monitoring would also be established, if requested by the RWQCB. The potential for water quality impacts could be minimized through mitigation measures similar to those used by the existing operation (Class II).

As with the other alternatives, the operation of machinery and heavy-duty vehicles pose the problem of petroleum contamination. The potential for spills and/or seepage of these pollutants would be significantly reduced by proper maintenance and storage of the equipment (Class II). Additionally, an emergency spill plan would be prepared and operators/employees trained in their appropriate course of action.

Mississippi Bar

Expansion of the Mississippi Bar operation to a production rate of 1,000 tons per hour may infringe upon existing environmental commitments. Erosion potential may increase as larger areas would be active at any given time. Surface runoff, excavation and processing impacts may be successfully controlled through expanded implementation of mitigation measures and environmental commitments already in place. Water quality impacts related to an expanded operation are potentially significant given the site's proximity to sensitive environs (Class II).

Yuba River Dredge Fields

The Yuba River dredge fields consist of approximately 8,500 acres of dredge tailings on either side of the Yuba River. The Western Aggregate operation is located on the south side of the river. Current facilities can accommodate a production rate of 1,000 tons per hour. The company has future plans to expand to 3,000 tons per hour, depending on market demand.

Processing includes both "wet" and dry operations. Wastewater from the wet side is piped to a series of settling ponds. Water flows through the ponds and fine material and other sediments settle to the bottom of the ponds. The "clean" water eventually drains into the Yuba River (some of the water percolates into the ground). The river water is monitored on a regular basis. Maintenance of the ponds includes occasional dredging of the settled material. New ponds are created as needed. To date, the only problems detected have been high silica concentrations. However, recent monitoring does not indicate a problem with silica. Surface runoff, excavation and processing impacts may be successfully controlled through expanded use of existing mitigation measures (Class II).

4.3.3 MITIGATION MEASURES

The potential for water quality impacts exists at all of the alternative source sites. Operations located adjacent to surface water would need to take additional precautions to prevent degradation of the resources. All of the operations would be subject to further evaluation and permitting actions by the RWQCB.

Planning of the selected operation should incorporate two critical aspects to minimize environmental impacts. The first employs best management practices (BMPs) throughout the construction/site preparation and mining/processing stages of the operation. Site reclamation is another aspect which should be considered early in the planning stages.

Mitigation measures for each alternative source are described below.

Middle Fork Sand and Gravel Deposits

Implementation of the project would increase the potential for contamination of runoff during the life of the project. The most likely contaminants would include sediments and petroleum residues. Sediment could result from soil disturbance and subsequent erosion. Petroleum residues (fuel, grease, etc.) could drip from machinery, vehicles, or fuel storage tanks. Several drainage control measures would need to be implemented to reduce the potential for degradation of local water quality.

Site preparation impacts could be reduced to less than significant by implementing the following measures:

- Construction of access routes and other activities that require removal of vegetation and disturbance of topsoils should be limited to low-flow periods in the dry months of the year (roughly May through October).

- Equipment and vehicles used during site preparation should be properly maintained and clean. Daily observation of all pieces should determine the potential for leaks or other problems. Maintenance, refueling, etc. shall be conducted in a specified area beyond the high water flow level (10-year floodplain).

- Interceptor channels, berms and temporary detention ponds should be designed for use during rainy season. These features shall be in place prior to stormy weather or snowmelt runoff. Appropriate surface runoff diversion structures shall be designed for the haul routes and the conveyor access route to prevent erosion of these passages.

- Where feasible, disturbed areas shall be revegetated. Established vegetation can reduce the impact of rainfall and slow the loss of surface soils.

Mining and excavation impacts are potentially significant. The change of the drainage course would have some lasting effects on the river hydrology. Water quality impacts caused by mining may be minimized through the implementation of the following measures:

- The project should include measures such as diversion berms, shallow ditches, etc., to divert overland drainage away from active harvesting areas. These structures would be temporary and should be destroyed following completion of excavation at each bar. Runoff would not be allowed to flow from an actively worked area and exit the site without passing through some form of erosion control measure. Possible measures could be used to minimize erosion, including construction of settling areas (no deeper than one foot), velocity dissipaters, small diversion berms, hay bale barriers, or other appropriate measures to slow runoff and facilitate deposition of silt and debris. Any settling areas would be of a temporary design and easily destroyed and recreated as the operation shifts location. The facilities would be properly maintained to preclude establishment of aquatic vegetation or the propagation of mosquitoes, and to insure ongoing effective control of erosion and sedimentation.

- The presence of fuel, grease, and similar products on the site in conjunction with the operation and maintenance of machinery is unavoidable, but would not occur to a degree that would pose a substantial risk to the surface or groundwater resources. A fuel tank spill containment structure should be used and designed to contain the content of the tank plus the precipitation associated with a 100-year 24-hour storm. Fuel, grease or similar products would not be stored in the harvesting areas. Equipment and vehicles used during mining should be properly maintained. Daily observation of all pieces should determine the potential for leaks or other problems. Repairs shall be conducted immediately. All maintenance work, repairs, refueling, etc., shall occur in a designated area beyond the high water level (10-year floodplain). Drainage structures would be in place at this location to divert flows away from the harvesting area and river channel.

- Reclamation of the area would be an ongoing process to the extent feasible. As soon as possible following harvesting of any given area, reclamation of that area would be implemented.

An ongoing reclamation program would enable the proponent and the regulatory agencies to evaluate the success of reclamation techniques in areas affected during early phases of the project prior to harvesting and reclamation of later phases. Subsequent harvesting and reclamation processes could be modified to optimize the process based on this information.

Processing plant impacts are generally limited to the disposal of wash water which includes fines such as clays and silts washed out of the crushing and screening units. The RWQCB states that ponds are typically not a problem as long as flows are not directly discharged to surface waters.

A monitoring program would be required at the Middle Fork site. Baseline conditions for the river would have to be established prior to any site preparation or excavation. Sampling locations would likely be required upstream and downstream of each excavation site.

Monitoring would be required on a monthly basis throughout the mining and reclamation stages. Daily observation of the settling ponds would be required including determination of daily flow and freeboard distance (monthly).

Semi-annual monitoring reports may be required by the RWQCB. These reports would include a description of monthly river sampling results and any observations related to floating or suspended matter, discoloration, bottom deposits and aquatic life.

Periodic inspections by the RWQCB area engineer would also occur. The results of the monitoring program and/or of the inspections may lead to changes in the Waste Discharge Requirements issued by RWQCB. The proponent would be responsible for implementing any changes.

Old Cool Quarry (Spreckles)

The activities at the Old Cool Quarry are expected to be contained within the quarry boundaries. The RWQCB should be involved in the review of wash water disposal operations. It may be possible that no permit action would be required even with the increased activity.

Maintenance of equipment and vehicles should be as discussed previously. Stormwater runoff should be diverted away from sources of potential contamination.

Additional site-specific mitigation would be developed should this site be selected.

Cool Quarry Amphibolite

Development of this site would require implementation of mitigation measures during site preparation. These measures would be similar to those described for the Middle Fork

alternative. Careful planning of the development of this site is important due to the steep topography and limited amount of space.

Operational impacts would be similar to the Old Cool Quarry. Once studies have determined appropriate location and design of settling ponds, the RWQCB should review the plans and determine the need for permitting. The RWQCB may require a monitoring and reporting program as discussed previously.

Bear River and Chevreaux Quarry

Mitigation of impacts at the Bear River-Chevreaux Quarry site would be similar to those described for the Middle Fork sand and gravel deposits. Due to the proximity of the operation to the Bear River, a monitoring program of the river water quality may be required. This program would include measuring constituent levels to assure compliance with RWQCB's non-degradation policy (see discussion of Middle Fork mitigation).

Mississippi Bar

Mitigation of impacts at Mississippi Bar would be required to avoid significant water quality impacts. The water quality impacts would be short-term and implementation of mitigation measures, such as those presented for the Middle Fork sand and gravel deposits alternative could reduce impacts to less than significant. A mitigation plan for this site should also continue consideration of the recreation and habitat qualities of the area.

Yuba River Dredge Fields

Expansion of a facility on the Yuba River would also require coordination with the RWQCB in regard to wash water disposal. It does not appear that site preparation or mining would create direct impacts on water quality.

If a site such as the Western Aggregate operation is selected, measures to ensure diversion of surface flows from active processing/mining areas would be implemented. Additional settling ponds may be designed. And, if required by the RWQCB, a program would be established to continue river water quality monitoring. Reports on the settling ponds would also be prepared, if determined necessary by the RWQCB or other regulatory agency.

4.4 AIR QUALITY

4.4.1 EXISTING CONDITIONS

Management of Airsheds and Pollutants of Importance

Air pollution control and airshed management are administered in the state of California by agencies of federal, state and local government. Both the federal and state agencies (the U.S. Environmental Protection Agency and the California Air Resources Board) have established ambient air quality standards, based on consideration of the health and welfare of the general public. For the purposes of air quality planning, the California Air Resources Board (CARB) designates 14 separate air basins within California. The concept of air basins recognizes the ability of winds to carry air pollutants throughout areas and the effects of topography and temperature inversions on such transport. Because of the nature of air, an air basin is not a precise physical division like a watershed, but a political construct for dealing with air problems that cross municipal boundaries. Each air basin is made up of one or more Air Pollution Control Districts (APCD) which function as the local regulatory agency.

Mountain Counties Air Basin. The Middle Fork sand and gravel deposits, Old Cool Quarry and Cool Quarry Amphibolite alternatives are all located within the Mountain Counties Air Basin. This air basin encompasses several counties located in the Sierra Nevada Mountain Range. It is bound to the west by the Great Valley, to the southwest by the San Joaquin Valley, and to the east by the Great Basin. Due to the meteorologic and climatic condition, the Mountain Counties Air Basin shares much of the same air pollution as the Sacramento Valley Air Basin.

El Dorado County Air Pollution Control District. Portions of the Middle Fork sand and gravel deposits and all of the Old Cool Quarry and Cool Quarry Amphibolite sites are located within the Mountain Counties Air Basin. All of El Dorado County constitutes the El Dorado County Air Pollution Control District (EDCAPCD), and this agency administers air quality regulations.

Placer County Air Pollution Control District. The portions of the Middle Fork sand and gravel deposits and Bear River and Chevreaux Quarry sites are located in the west central portion of the Mountain Counties Air Basin. All of Placer County constitutes the Placer County Air Pollution Control District (PCAPCD). The north side of the Middle Fork river lie within the PCAPCD, and the south side of the river lies within EDCAPCD, as the county boundaries are located in the middle of the Middle Fork of the American River.

Nevada County Air Pollution Control District

The Bear River and Chevreaux Quarry lie within the northern portion of the Mountain Counties Air Basin. All of Nevada County constitutes the Nevada County Air Pollution

Control District (NCAPCD). The western portion of Lake Combie lies within Nevada County, while the eastern portion lies within the PCAPCD. However, since the majority of Chevreaux Quarry's mining activity is operated in the PCAPCD, the NCAPCD does not require compliance to their permitting regulations.

Sacramento Valley Air Basin. The Yuba River Dredge Fields and Mississippi Bar alternatives lie within the Sacramento Valley Air Basin. This basin is part of the northern portion of the Great Valley and extends into the neighboring mountain ranges. It is bounded on the west by the Coast Range, on the north and east by the Cascade Range and the Sierra Nevada Range, and on the south by the San Joaquin Valley Air Basin. The Sacramento Basin covers a region which, because of similar meteorological and geographical conditions, shares the same air and, hence, the same air pollution problems as these other regions.

Sacramento Metropolitan Air Quality Management District. Mississippi Bar lies within the southeast portion of the Sacramento Valley Air Basin. Yolo County, Sacramento County southwest Placer County and northern Solano County currently comprise the Sacramento Metropolitan Air Quality Management District (SMAQMD). The SMAQMD is the local agency responsible for the planning and maintenance of state and federal air quality standards. The SMAQMD 1991 Air Quality Plan was approved by the District Board on July 17, 1991.

Feather River Air Quality Management District. The Yuba River Dredge Fields lie within the south-central portion of the Sacramento Valley Air Basin. Yuba County and Sutter County have formed the Feather River Air Quality Management District (FRAQMD). There is no current air quality management plan in place. However, a draft Air Quality Attainment Plan is currently under review by both counties.

Pollutants of Concern. The principal air pollutant of concern to the Sacramento Valley Air Basin is ozone, the main constituent of photochemical smog. Ozone is not released directly into the atmosphere. Rather, it is a secondary pollutant resulting from a complex series of photochemical reactions. These reactions occur when precursor compounds, such as hydrocarbons and nitrogen oxides (NO_X) are mixed by light winds and heated by the sun. Hydrocarbon emissions represent a compound of reactive organic gases (ROGs), which results from evaporation of petroleum products. Nitrogen oxide emissions result from combustion of petroleum products.

ROGs and NO_X, measured in tons per day, are emitted into the air from a variety of sources. These sources are generally grouped into two main categories: stationary and mobile. Stationary sources consist of major industrial, manufacturing and processing plants ("point" sources) and commercial/industrial facilities which individually emit only small quantities of pollutants but collectively result in significant emissions ("area" sources). Mobile sources consist of on-road motor vehicles, including automobiles, trucks and buses; and off-road vehicles such as construction equipment, farm tractors, trains, ships and aircraft.

Health Considerations and Air Quality Standards

Health Considerations. The health effects of ozone include aggravation of respiratory illnesses, chronic heart and lung disorders and some anemias. Ozone can also harm normal, healthy adults in concentrations found regularly in various parts of the state. The effects often include nausea, headaches, eye irritation, dizziness, throat pain, breathing difficulty and coughing. The health effects caused by combined concentrations of certain sulfur oxides and ozone are more severe than those caused by greater concentrations of either pollutant alone.

Carbon monoxide (CO) is another, though less pervasive, primary pollutant. CO is emitted directly into the atmosphere, and is generally dispersed from the emission source and diluted through mixing. CO problems are usually localized and result from a combination of high traffic volumes and significant traffic congestion. CO is most often a problem in winter months as a result of radiation inversion, in which air near the ground cools in the evening by radiative processes while air aloft remains warm.

The inversions, coupled with calm conditions, cause "hot spots" near the emission source due to poor dispersive capacity during winter nights. The inversions usually burn off in the morning. CO levels are a public health concern due to the greater affinity of the CO molecules, resulting in reduce 0_2 transport in the blood. Station and national standards were established to keep the CO-HgB concentration below levels that will harm cardiovascular and central nervous systems.

Air Quality Standards. As mandated by the Clean Air Act of 1977 Amendments (Federal Act), EPA has established National Ambient Air Quality Standards (NAAQS) for a variety of pollutants including ozone and CO. These standards, shown in Table 4.4-1, have been set at concentrations designed to protect the health and welfare of people most susceptible to respiratory distress, such as the acute and/or chronically ill, young children, the elderly and persons engaged in strenuous work. The Federal Act requires each state to develop a State Implementation Plan detailing the pollution control measures necessary to attain the adopted standards. Areas that do not meet these standards for any or all constituents are designated as "nonattainment" areas. National standards set for PM_{10} are classified into three groups. Areas designated into Group 1 have a strong likelihood of violating the standard; areas designated as Group 2 indicates uncertain attainment of the standard; and Group 3 is not classified under Group 1 or Group 2.

State air quality standards have been established in California by the State Air Resources Board (ARB). As indicated in Table 4.4-1, these standards are generally more stringent than those established by EPA. Under the California Clean Air Act of 1988 (Sher Bill), the ARB is required to establish criteria for identifying air basins which have not attained state air quality standards. Air basins which are designated as nonattainment areas, and which,

TABLE 4.4-1
AIR QUALITY STANDARDS

Pollutant	Averaging Time	California Standards[1]		National Standards[2]		
		Concentration[3]	Method[4]	Primary[3,5]	Secondary[3,4,6]	Method[7]
Ozone	1 Hour	0.09 ppm (180 µg/m3)	Ultraviolet Photometry	0.12 ppm (235 µg/m3)	Same as Primary Std.	Ethylene Chemiluminescence
Carbon Monoxide	8 Hour	9.0 ppm (10 mg/m3)	Non-dispersive infrared Spectroscopy (NDIR)	9.0 ppm (10 mg/m3)		Non-dispersive infrared Spectroscopy (NDIR)
	1 Hour	20 ppm (23 mg/m3)		35 ppm (40 mg/m3)		
Nitrogen Dioxide	Annual Average	---	Gas Phase Chemiluminescence	0.053 ppm (100 µg/m3)	Same as Primary Std.	Gas Phase Chemiluminescence
	1 Hour	0.25 ppm (470 µg/m3)		---		
Sulfur Dioxide	Annual Average	---	Ultraviolet Fluorescence	80 µg/m3 (0.03 ppm)	---	Pararosoaniline
	24 Hour	0.05 ppm[8] (131 µg/m3)		365 µg/m3 (0.14 ppm)	---	
	3 Hour	---		---	1300 µg/m3 (0.5 ppm)	
	1 Hour	0.25 ppm (655 µg/m3)		---	---	
Suspended Particulate Matter (PM_{10})	Annual Geometric Mean	30 µg/m3	Size Selective Inlet High Volume Sampler and Gravimetric Analysis	---	---	---
	24 Hour	50 µg/m3		150 µg/m3	Same as Primary Stds.	Inertial Separation and Gravimetric Analysis
	Annual Arithmetic Mean	---	---	50 µg/m3		
Sulfates	24 Hour	25 µg/m3	Turbidimetric Barium Sulfate	---	---	---
Lead	30 Day Average	1.5 µg/m3	Atomic Absorption	---	---	Atomic Absorption
	Calendar Quarter	---		1.5 µg/m3	Same as Primary Std.	
Hydrogen Sulfide	1 Hour	0.03 ppm (42 µg/m3)	Cadmium Hydroxide Stractan	---	---	---
Vinyl Chloride (chloroethane)	24 Hour	0.010 ppm (26 µg/m3)	Tedlar Bag Collection, Gas Chromatography	---	---	---
Visibility Reducing Particles[9]	8 Hour (10 a.m.-6 p.m. PST)	In sufficient amount to produce an extinction coefficient of 0.23 per kilometer due to particulates when the relative humidity is less than 70 percent. Measurement in accordance with ARB method V.		---	---	---

(Footnotes are listed on the following page.)

NOTES

1. California standards for ozone, carbon monoxide, sulfur dioxide (1 hour), nitrogen dioxide, suspended particulate matter - PM_{10} and visibility-reducing particulates, are values that are not to be exceeded. The sulfur dioxide (24-hour), sulfates, lead, hydrogen sulfide and vinyl chloride standards are not to be equaled or exceeded.

2. National standards, other than ozone and those based on annual averages or annual arithmetic means, are not to be exceeded more than once a year. The ozone standard is attained when the expected number of days per calendar year with maximum hourly average concentrations above the standard is equal to or less than one.

3. Concentration is expressed first in units in which it was promulgated. Equivalent units given in parentheses are based upon a reference temperature of 25° C and a reference pressure of 760 mm of mercury. All measurements of air quality are to be corrected to a reference temperature of 25° C and a reference pressure of 760 mm of mercury (1,013.2 millibar); ppm in this table refers to ppm by volume, or micromoles of pollutant per mole of gas.

4. Any equivalent procedure which can be shown to the satisfaction of the Air Resources Board to give equivalent results at or near the level of the air quality standard may be used.

5. National Primary Standards: The levels of air quality necessary, with an adequate margin of safety to protect the public health. Each state must attain the primary standards no later than three years after that state's implementation plan is approved by the Environmental Protection Agency.

6. National Secondary Standards: The levels of air quality necessary to protect the public welfare from any known or anticipated adverse effects of a pollutant. Each state must attain the secondary standards within a "reasonable time" after the implementation plan is approved by the EPA.

7. Reference method as described by the EPA. An "equivalent method" of measurement may be used but must have a "consistent relationship to the reference method" and must be approved by the EPA.

8. At locations where the state standards for ozone and/or total suspended particulate matter are violated. National standards apply elsewhere.

9. This standard is intended to limit the frequency and severity of visibility impairment due to regional haze and is equivalent to a ten-mile nominal visual range when relative humidity is less than 70 percent.

like the Sacramento Valley Air Basin, receive or contribute to transported air pollutants, were required to submit to the ARB a plan for attaining state standards by June 30, 1991.

Middle Fork Sand and Gravel Deposits. The Middle Fork sand and gravel deposit alternative is located in the PCAPCD and the EDCAPCD; both are within the Mountain Counties Air Basin. As depicted in Figure 4.4-1, both districts have been designated as a nonattainment area for ozone. In addition, Placer County has been designated as a nonattainment area for particulate matter (PM_{10}).

Old Cool Quarry. The Old Cool Quarry alternative is located within the EDCAPCD within the Mountain Counties Air Basin. As shown in Figure 4.4-1, the EDCAPCD has been designated as a nonattainment area for PM_{10}.

Cool Quarry Amphibolite. This alternative is adjacent to Old Cool Quarry and under the same air pollution control district.

Bear River and Chevreaux Quarry. The Bear River and Chevreaux Quarry is located in the PCAPCD and the NCAPCD. Both districts are within the Mountain Counties Air Basin. As depicted in Table 4.4-2, Placer County has been designated as a nonattainment area for ozone. As depicted in Table 4.4-3, Nevada County has been designated as a nonattainment area for PM_{10}.

Yuba River Dredge Fields. The Yuba resources area is located in the FRAQMD within the Sacramento Valley Air Basin. As depicted in Table 4.4-2 and Table 4.4-3, Yuba County has been designated as a nonattainment area for ozone and PM_{10}.

Mississippi Bar. Mississippi Bar is located in the SMAQMD within the Sacramento Valley Air Basin. As depicted in Tables 4.4-2 and 4.4-3, Sacramento County has been designated as a nonattainment area for ozone and PM_{10}.

4.4.2 IMPACT ANALYSIS

Methodology and Significance Criteria

The following air quality impact analysis is based on a preliminary review of the air quality impacts which can be anticipated from implementation of gravel extraction, processing and transport.

For the purposes of this report, any project-generated emission of pollutants designated nonattainment by the ARB that cannot be offset elsewhere in the air basin are considered significant air quality impacts. In addition, any predicted project-induced exceedence of Federal or State Ambient Air Quality Standards was also considered a significant air quality impact.

TABLE 4.4-2
ATTAINMENT STATUS FOR OZONE FOR MOUNTAIN COUNTIES AND SACRAMENTO VALLEY AIR BASIN

	OZONE	
	NAAQS	CAAQS
MOUNTAIN COUNTIES AIR BASIN		
El Dorado County	UN[1]	NON[2]
Placer County	NON	NON
Nevada County	UN	UN
SACRAMENTO VALLEY AIR BASIN		
Sacramento County	NON	NON
Yuba County	NON	NON

[1] UN = Unclassified
[2] NON = Nonattainment

TABLE 4.4-3
ATTAINMENT STATUS FOR PM_{10} FOR MOUNTAIN COUNTIES AND SACRAMENTO VALLEY AIR BASIN

	PM_{10}	
	NAAQS	CAAQS
MOUNTAIN COUNTIES AIR BASIN		
El Dorado County	Group 3	NON[1]
Placer County	Group 3	UN[2]
Nevada County	Group 3	NON
SACRAMENTO VALLEY AIR BASIN		
Sacramento County	Group 3	NON
Yuba County	Group 3	NON

[1] NON = Nonattainment
[2] UN = Unclassified

The proposed project would not result in any permanent point source air pollutant emission sources. The project would, however, result in short-term mobile and point source air quality impacts during the extraction, processing and transport of quarry material. Short-term emissions would also occur during the project construction phase. Depending on which source is ultimately selected, short-term construction phase could include assembly of excavation and processing facilities for those sources which are not currently operational as mines, and developing temporary roads as haul routes, or to access and assemble temporary conveyor systems. These aspects of the project's potential impacts on air quality are discussed in more detail below.

Extraction and Processing. Gravel extraction and processing would result in both particulate and combustion emissions at the extraction site. Extraction itself would be accomplished either by blasting or by dragline. Blasting would be utilized at the quarry sites. Blasting results in large quantities of dust and particulate generation. Because of the nature of blasting, the short-term localized exceedence of State Ambient Air Quality Standard for PM_{10} is anticipated. Consequently, where blasting is required, project generated dust and particulate emissions would be considered a significant short-term impact (Class II) on air quality.

Dragline techniques would be used for gravel extraction from those sources located in-stream. Because the majority of gravel extracted by dragline would come from below the high water line, dust and particulate emissions would be substantially less compared to blasting. It is not anticipated that localized exceedences of the PM_{10} standards would occur during gravel extraction by dragline. Consequently, related impacts would be considered not significant.

Dragline techniques would, however, produce combustion emissions from the engine used to power the dragline. Considered alone, however, dragline combustion emissions would be insignificant compared to basin-wide combustion emissions.

Processing procedures would immediately follow extraction. For this analysis, it is assumed that processing emissions will be the same for all alternatives. Depending on the source selected, processing would either be conducted at the source site, as would be the case with existing quarry operations, or at another location. If processing is conducted at another location, additional combustion emissions associated with transport would result. In addition, dust emissions associated with the added loading and unloading prior to and after transport operations may result.

During processing of the aggregate materials, the majority of emissions are released as particulates. The type, size, and quantity of particulates released depends on the composition of the aggregate itself, the type of processing used and the machinery used. Specific information concerning the make-up of quarry material is unavailable at this time. However, it is assumed that the sand and gravel processing system ultimately utilized would have standard dust suppression systems incorporated in its design. These suppression

systems consist of water spray nozzles and wet screens located at various locations throughout the plant. According to a previous study conducted for the Yolo-Solano Air Pollution Control District, a 750,000 ton per year plant can produce approximately 19 tons per year of particulates (Quad Consultants, 1989). This emission level is below the level which requires application of New Source Review Rules in the Yolo-Solano APCD. Nevertheless, for those districts which are currently classified as nonattainment for PM_{10}, the incremental addition of particulate matter as a result of project generated processing operations would be considered significant.

Combustion emissions could also be generated both on and off site depending on the location of the power source for the crusher. If power generation by combustion is located in a district where ozone is in nonattainment, air quality impacts would be considered significant.

Transport. Gravel transport would occur either by truck, train or conveyor. Transport by truck or train would result in short-term combustion emissions. Large diesel trucks emit an average of 0.5 pounds of NO_X for each gallon of fuel burned (EPA AP 42, 1985), but emits minor amounts of CO as compared gasoline powered engines. Gasoline powered trucks produce about one pound of CO and lesser amounts of NO_X and hydrocarbons for ever gallon of fuel burned (PRC Toups, February 1983). Total emissions would depend on the overall vehicle miles traveled for the gravel site selected. Combustion emissions would be considered significant short-term for those districts currently designated nonattainment for ozone, CO and NO_X.

Conveyor systems also produce combustion emissions from generators used to power the electric conveyors. Conveyor systems also produce significant amounts of dust and particulate emissions at loading and transfer points. Dust may also become airborne due to vibration of the belt as it passes over rollers or around drums, as a result of spillage, and at obstructions such as ventilation doors of flaps which the belt may have to negotiate. Overall dust emissions resulting from use of conveyors would be considered a short-term significant impact (Class II) subject to mitigation.

Comparison Summary. In general, project-generated emissions are considered significant short-term impacts (Class I) due to the existing nonattainment status of the effected air basins. Extraction and processing emissions between alternative sources considered would for the most part be relatively equal with the exception of PM_{10} which would be greater for those sources requiring blasting. The greatest variable in terms of air pollutant emissions would result from transport. The transportation air quality impacts associated with the alternative sources considered would be proportional to the travel distance required. Consequently, those sources nearest the proposed dam site would have the least overall air quality impacts and those sources farthest from the dam site would have correspondingly greater air quality impacts.

4.4.3 MITIGATION MEASURES

Project generated emissions were determined to result in significant short-term impacts on air quality. The following measures would assist in reducing project generated emissions to the greatest extent feasible.

- Where feasible, project vehicles should be fitted with emission reduction equipment.
- Water trucks should be used regularly to reduce dust and particulate generation at quarry sites, construction sites and along non-paved travel roads.
- Operations should be restricted or banned on days when air quality violations are expected.
- Conveyor systems should be carefully planned and installed.
- Dust adhering to conveyor belt surfaces should be removed and collected.
- Spillage along the length of the conveyor should be periodically removed.
- Processing facilities should be equipped with the best available control technology for the suppression of dust. This should include extensive use of screens and water.
- Offset project-generated emissions elsewhere in the air basin if possible.

4.5 FISH, VEGETATION, AND WILDLIFE

4.5.1 EXISTING CONDITIONS

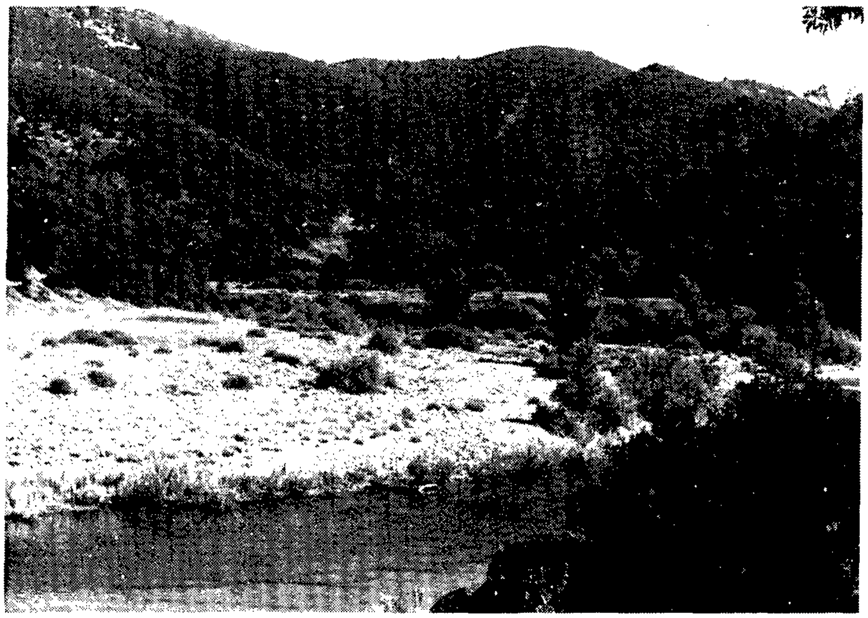
Middle Fork Sand and Gravel Deposits.

Approximately 6,760,000 cubic yards of aggregate would be required for the production of the 5,000,000 cubic yards of material necessary for the roller compacted concrete (RCC) dam. The estimated cumulative volume of sand and gravel available within the gravel bars of the Middle Fork of the American River between Mammoth Bar and Cherokee Bar is approximately 8,605,000 cubic yards. Including the estimated 986,250 cubic yards of material also available within the river channel, acquisition of gravel from the bars would result in a net loss of approximately 72.3 percent of the present volume.

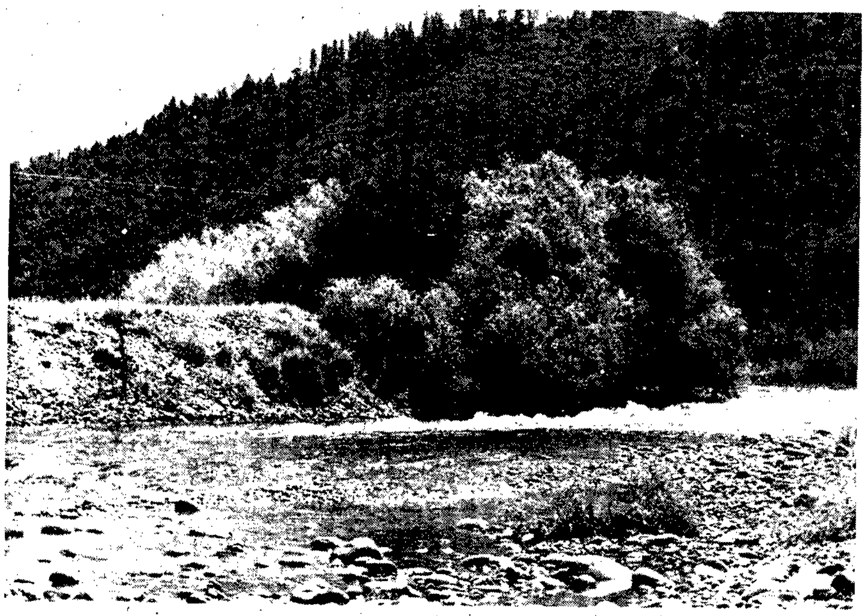
The ten gravel bars, located between Mammoth Bar at approximate river mile (RM) 25.5 (elevation 600 feet, msl) and Cherokee Bar at RM 31.4 (elevation 700 feet, msl), project between 5 to 25 feet above the normal water surface elevation (WSE). Along straight sections of the river and at point bars, the elevation gradient from the water's edge is gradual (Figure 4.5-1); however, along cut banks opposite of point bars, the bank slopes are steep, and the first terrace is generally 8-12 feet above the water surface (Figure 4.5-1). The interior of the bars transition from the water's edge to the canyon escarpment in a series of terraces, each exhibiting more xeric vegetation with elevational increase (Figure 4.5-2). A similar gravel bar physiography and corresponding distribution and transition of vegetation has been described in other riverine situations in California (Faber et al. 1989; Harris 1988; Lisle 1988; McBride and Strahan 1984; and Strahan 1984), and other western states (Minshall et al. 1989; Fyles and Bell 1986). The principal vegetative cover types inhabiting the gravel bars are *Riparian Shrub/Scrub* and *Gravel Bar Scrub* and are described below.

Vegetative Cover Types

Riparian Shrub/Scrub. Typically, the riverward edge of the bars are vegetated in discontinuous stringers of riparian shrub/scrub composed principally of white alder, various willow species, Oregon ash, Fremont cottonwood, and locust. Understory vegetation includes blackberries, hedge-nettle, wild grape, wild rose, creek monkeyflower. Table 4.5-1 is a listing of the species identified within the various vegetative communities potentially impacted by the proposed project. Maximum canopy heights seldom exceed 20-30 feet, and the width or the stringers vary from 10 to 60 feet. The riparian shrubs become established during spring and summer at the low flow margin of the gravel bar in response to the need for year-round moisture. Alders and willows tend to occur in areas immediately adjacent to the stream, while oaks and conifers are associated with the outer floodplain limits (Harris 1988). As growth proceeds, the shrubs stabilize bank materials by adding root strength and reducing local shear stress through increased roughness, which both enables the shrubs to



a. Typical display of gradual slope from river edge to canyon escarpment (Texas bar).



b. Characteristic cut bank at Hoosier Bar. Note height of bank and steep slope to primary terrace.

Biological Resources - Middle Fork American Riv

FIGURE 4.5-1

TABLE 4.5-1
TYPICAL VEGETATION, BY COVER TYPES, OBSERVED IN THE AMERICAN RIVER CANYON

SPECIES		COVER TYPES				
COMMON NAME	*SCIENTIFIC NAME*	OAK[1]	CHA[1]	BAR[2]	PINE[1]	RIP[2]
TREES AND SHRUBS						
Ponderosa pine	*Pinus ponderosa*	X	X			
Knobcone pine	*P. attenuata*					
Sugar pine	*P. lambertiana*					
Digger pine	*P. sabiniana*					
Douglar fir	*Pseudotsuga menziesii*					
White fir	*Abies concolor*					
Incense cedar	*Calocedrus decurrens*					
Oregon ash	*Fraxinus latifolia*					X
Locust	*Robinia pseudo-acacia*					X
White alder	*Alnus rhombifolia*					X
Fremont cottonwood	*Populus fremontii*					X
Willows	*Salix sp.*					X
Wild grape	*Vitis californica*					X
California buckeye	*Aesculus californica*	X	X	X		
Bigleaf maple	*Acer macrophyllum*	X				X
Poison oak	*Toxicodendron radicans*	X	X	X		
Coyotebush	*Baccharis pilularis*			X		
Elderberry	*Sambucus mexicani*			X		X
Black walnut	*Juglans hindsii*			X		X
Interior live oak	*Quercus wizlensii*	X	X	X		
Canyon live oak	*Q. chrysolepis*					
Black oak	*Q. kelloggii*					
Valley oak	*Q. lobata*					
Blue oak	*Q. douglasii*					
Tan-oak	*Lithocarpus densiflora*					
Sandbar willow	*Salix hindsii*					X
Acacia	*Acacia sp.*					X
Mulefat	*Baccharis viminea*					X
California brickellbush	*Brickellia californica*			X		
Dusky willow	*Salix melanopsis*					X
Coffeeberry	*Rhamnus sp.*					
Pacific madrone	*Arbutus menziesii*					
Western redbud	*Cercis occidentalis*					
California hazelnut	*Corylus rostrata*					
Saltbush	*Atriplex sp.*					

TABLE 4.5-1 (CONTINUED)

SPECIES		COVER TYPES				
COMMON NAME	*SCIENTIFIC NAME*	OAK[1]	CHA[1]	BAR[2]	PINE[1]	RIP[2]
TREES AND SHRUBS (CONTINUED)						
California bay	*Umbellularia californica*					
Buck brush	*Ceanothus cuneatus*					
Chamise	*Adenostoma fasciculatum*					
Western mountain-mahogany	*Cercocarpis betuloides*					
Flannelbush	*Fremontodentron californica*					
Toyon	*Heteromeles arbutifolia*					
Snowberry	*Symphoricarpos sp.*					
Oregon golden-aster	*Chryopsis oregona*			X		
Manzanita	*Arctostaphylos sp.*	X	X	X		
GRASSES AND FORBS						
Spike moss	*Selaginella hansenii*			X		
Carolina geranium	*Geranium carolinianum*		X	X		
Stocksbill	*Erodium spp.*		X	X		
Turkey mullein	*Eremocarpus setigeris*			X		
Spurge	*Euphorbia spp.*			X		
Durango root	*Datisca glomerata*			X		
Field mustard	*Brassica campestris*			X		
Black mustard	*Brassica nigra*			X		
Shepard's purse	*Capsella bursa-pectoris*			X		
Catchfly	*Silene spp.*			X		
Miner's lettuce	*Montia perfoliata*			X		
Milkweed	*Asclepias cordifolia*			X		
Gilia	*Gilia capitata*			X		
Popcorn flower	*Plagiobothrys spp.*		X	X		
Creek monkey flower	*Mimulus guttatus*			X		
Common mullein	*Verbascum thaspus*		X	X		
Gay penstemon	*Penstemon laetus*	X	X	X		
Foothill penstemon	*Penstemon speciosus*	X	X	X		
Bluecurls	*Trichostema oblongum*		X	X		
White hedge-nettle	*Stachys albens*					X
Sage	*Salvia spp.*			X		
Live-forever	*Dudleya spp.*		X	X		
Indian rhubarb	*Peltiphyllum peltatum*					X
California blackberries	*Rubus vitifolius*					X
California wild rose	*Rosa california*	X				X
Lupine	*Lupinus stiversii*	X	X	X		

TABLE 4.5-1 (CONTINUED)

SPECIES		COVER TYPES				
COMMON NAME	*SCIENTIFIC NAME*	OAK[1]	CHA[1]	BAR[2]	PINE[1]	RIP[2]
GRASSES AND FORBS (CONTINUED)						
Spanish broom	*Spartium junceum*		X	X		
Bird's foot trefoil	*Lotus micranthus*		X	X		
California vetch	*Vicia californica*		X	X		
Pacific sanicle	*Sanicula spp.*		X	X		
Mule-ears	*Wyethia spp.*		X	X		
Aster	*Aster spp.*	X	X	X		
Fleabane	*Erigeron divergens*	X	X	X		
Mugwort	*Artemisia douglasiana*			X		X
Bull thistle	*Cirsium vulgare*	X	X	X		
Yellow star thistle	*Centaurea melitensis*	X	X	X		X
Blue-eyed grass	*Sisyrinchium bellum*			X		
Sedge	*Carex spp.*					X
Ripgut brome	*Bromus diandrus*	X	X	X		
Foxtail barley	*Hordeum jubatum*	X	X	X		
Italian ryegrass	*Lolium multiflorum*	X	X	X		
Cocklebur	*Xanthium strumarium*			X		X
Buckthorn plantain	*Plantago lanceolata*			X		
Fiddleneck	*Amsinckia intermedia*		X	X		
Smartweed	*Polygonum lapathifolium*			X		X
Horsetail fern	*Equisteum sp.*			X		X
Yarrow	*Achillea lanulosa*			X		X
Globe lily	*Calochortus albus*	X	X	X		
Indian paintbrush	*Castilleja sp.*		X	X		
Monkeyflower	*Mimulus sp.*			X		
Wild oak	*Avena fatua*	X	X	X		
Dallis grass	*Paspalum dilatatum*			X		X
Red brome	*Bromus rubra*	X	X	X		
Bottlebrush squirreltail	*Sitanion hystrix*	X	X	X		

Notes:

OAK -- Associated with North Slope Oak Forests and/or South Slope Oak Woodlands
CHA -- Associated with Chaparral Community
BAR -- Associated principally with Gravel Bar Scrub Community
PIN -- Associated with Pine Forest Community
RIP -- Associated with Riparian Shrub/Scrub Community

[1] Based on reconnaissance surveys conducted by U.S. Fish and Wildlife Service.
[2] Based on reconnaissance surveys conducted by Fugro-McClelland (West), Inc.

a. View of Texas Bar of characteristic vegetation patterns found on gravel bars. Note riparian shrub/scrub stringers at river edge, transitioning to sparsely vegetated gravel bar scrub, to dense grassland/ruderal vegetation, and ultimately to canyon escarpment vegetated in oak woodland cover.

b. Typical view of sparsely vegetated gravel bar (areal coverage less than 30 percent).

Biological Resources - Middle Fork American River

FIGURE 4.5-2

withstand high winter flows and facilitates the deposition and retention of finer particles (Lisle 1988).

The riparian stringer community most closely resembles the description of the *Great Valley Willow Scrub* described in the California Natural Diversity Database (CNDD) system (Holland 1986) (Appendix 6.1).

Under the U.S. Fish and Wildlife Service classification system (Cowardin et al. 1979), the riparian stringers would be categorized as a *Palustrine Scrub-Shrub* wetland based on a greater than 30 percent areal coverage of persistent emergents (cattails, tules, etc.), trees, shrubs, or aquatic mosses. However, for this report, the cover type will be referred to as Riparian Shrub-Scrub.

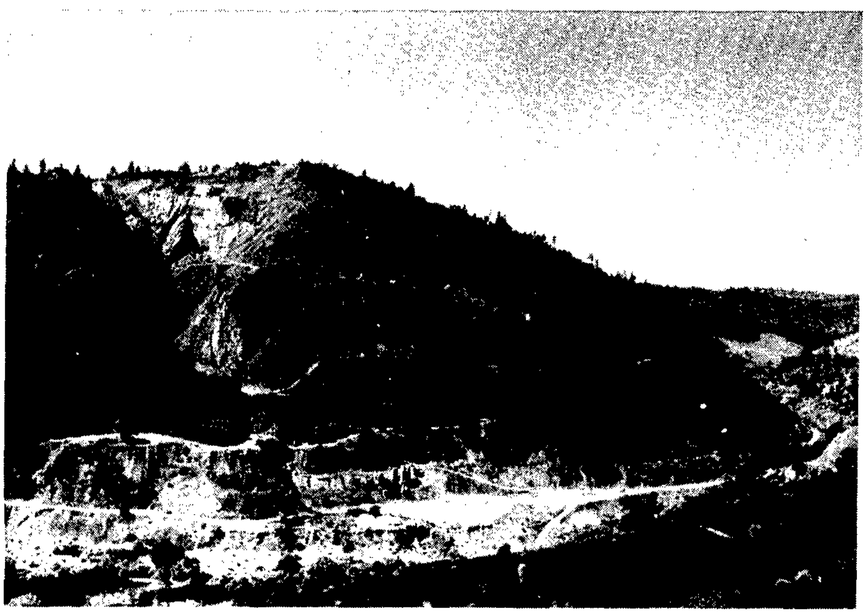
Gravel Bar Scrub. As the gravel bars increase in elevation from the low-flow margin to the canyon escarpment, the vegetation transitions from hydric to mesic species in response to a reduction in water availability. This reduction results from the increasing depth to the water table, low water storage capacity of the coarse substrate, and reduced capillary action of the substrate (Minshall et al. 1989). Coarse textured soils also exhibit decreased fertility as a result of low nutrient exchange capacity. Additionally, gravel and cobble substrate decrease the ability of seedlings to establish root systems to exploit whatever water and nutrients are available (Fyles and Bell 1986).

The areal extent of vegetation within the interior of the gravel bars varies from sparsely (Figure 4.5-2) to moderately vegetated (Figure 4.5-3). The species composition is generally more mesic in character and was often dominated by California brickellbush. Other characteristic species include Oregon golden-aster, mugwort, black mustard, yellow star thistle, Spanish broom, an occasional pine, common mullein, lupines, storksbill, geranium, etc. (see Table 4.5-1).

A corresponding community classification has not been assigned to the cover type occupying the interior of the bars under the CNDD system (Holland 1986). However, under the U.S. Fish and Wildlife Service classification system (Cowardin et al. 1979), these interior areas would be categorized as either *Riverine Unconsolidated Shore (Vegetated)* or *Riverine Unconsolidated Shore*.

In total, the ten bars encompass approximately 206 acres. An estimate of the areal coverage of the two dominant vegetative cover types was attempted using aerial photographs of the gravel bars. However, much of the herbaceous growth was indistinct (similar difficulties in mapping gravel bar vegetation were encountered by Nelson and Nelson (1984) along the Sacramento River). To address this concern, visual estimates of vegetative cover were made in the field by a team consisting of staff biologists from the U.S. Fish and Wildlife Service, Corps of Engineers, and the California Department of Water Resource's biological consultant. Based on the consensus visual estimates and planimetric analysis of the area of each bar performed by the U.S. Army Corps of Engineers (1991), an estimate of the areal

a. Typical view of moderately vegetated gravel bar (areal coverge less than 30 percent).



b. View of Old Cool Quarry (Spreckles) site (left) and Cool Quarry Amphibolite (center). Vegetative cover is typical of oak woodlands found on site.

Biological Resources - Middle Fork American River

FIGURE 4.5-3

extent of the cover types was made. Of the 206 acres encompassed by the gravel bars, approximately 98.2 acres (47.6 percent) was exposed substrate; 75 acres (36 percent) was categorized as *Gravel Bar Scrub*; and, 34 acres (16.5 percent) were determined to be *Riparian Shrub-Scrub* (Table 4.5-2).

In addition to the streamside vegetation communities, the project would also impact upland communities through the construction of access roads and the aggregate conveyor system. Quantification of the vegetative cover types was determined using Wildlife Habitat Cover Type maps prepared by the U.S. Army Corps of Engineers based on vegetative community mapping conducted by U.S. Fish and Wildlife Service (USFWS). The proposed project features were superimposed on the cover type maps and the area impacted was determined using a rolling map measurer to determine linear distances and a planimeter to determine area. The width of access roads and conveyor route rights-of-way were assumed to be 25 feet. The upland communities included the following cover types:

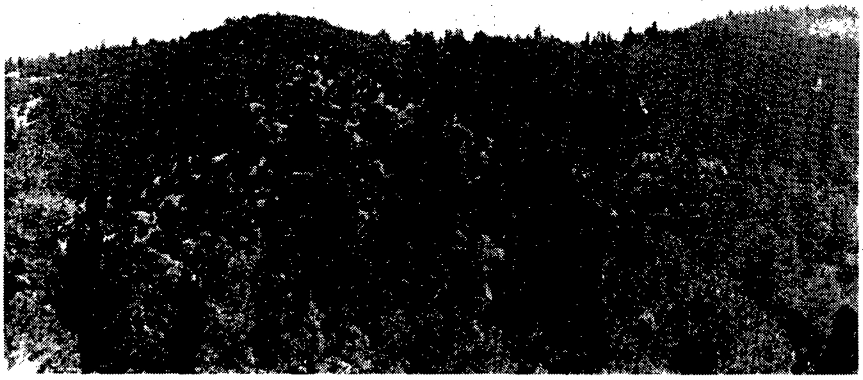
Oak Woodlands. The gravel bars transition to upland cover types at the canyon escarpment. Vegetation on these slopes is similar in composition; however, USFWS (1991) noted that canopy closure varies with aspect. As a result, drier south- and southwest-facing slopes tend to exhibit woodland characteristics with open to moderately open canopies, while north and northeast-facing slopes are more forest-like with canopy coverage exceeding 50 percent. USFWS (1991) has described these cover types as *South Slope Oak Woodland* (Evergreen Hardwood Woodland) and *North Slope Oak Forest* (Evergreen Hardwood Forest), respectively. Typical species associated with these cover types include interior live oak, black oak, canyon live oak, California bay, Douglas fir, ponderosa pine, digger pine, California buckeye, bigleaf maple, and an occasional madrone (Figure 4.5-3). The understory is composed principally of toyon, manzanita, coffeeberry, buckbrush, poison oak, yerba santa, coyotebush, and shrubs of the aforementioned tree species (Table 4.5-1). These cover types are similar in composition and distribution to the Interior Live Oak Woodland community described by Holland (1986) and the Interior Live Oak/Toyon cover type described by Allen et al. (1989) (Appendix 6.2).

Chaparral. Along the upper canyon slopes (Figure 4.5-4), particularly on south-facing slopes with limestone, serpentine, gabbro, or other highly mineralized soils, the oak woodlands transition into chaparral, which is dominated by chamise, white-leaf manzanita, buck brush, toyon, yerba santa, and shrub-sized interior live oak and canyon live oak (Table 4.5-1). These stands are typical of the Northern Mixed Chaparral community described by Holland (1986) (Appendix 6.2).

Conifer Forest. Scattered stands of conifer forest are found within the proposed project area along both sides of the river (Figure 4.5-4). U.S. Fish and Wildlife Service (1991) recorded both monotypic and mixed conifer stands within the inundation zone of the proposed flood control dam which were composed variously of Douglas fir, ponderosa pine, digger pine, and knobcone pine (Table 4.5-1). Rundel et al. (1988) reported that within the northern Sierra, the ponderosa pine forest occurs from about the 975-foot elevation to over 5,860 feet. At

TABLE 4.5-2 ESTIMATED AREAL COVERAGE OF VEGETATION, BY COVER TYPE, ON MIDDLE FORK OF THE AMERICAN RIVER GRAVEL BARS BETWEEN RIVER MILES 25.5 AND 31.4

BAR	COVER TYPE ACREAGE			TOTAL ACREAGE
	Riparian	*Gravel Bar Scrub*	*Exposed*	
Cherokee Bar	3.4	23.9	6.8	34.1
Poverty Bar	5.3	2.7	18.5	26.5
Philadelphia Bar	2.0	4.2	14.6	20.8
Maine Bar	0.6	2.2	2.9	5.7
Buckeye Bar	5.1	10.1	10.1	25.3
Hoosier Bar	1.5	5.2	8.2	14.9
Kennebec Bar	1.7	4.1	10.7	16.5
Browns Bar	1.4	9.7	2.8	13.9
TexasBar	9.3	9.4	7.9	26.6
Mammoth Bar	3.3	3.3	15.7	22.3
TOTAL	33.55	74.8	98.2	206.6



a. Typical view of chaparral cover type found on upper slopes.

b. View of conifer forest stand found in project area.

the lower elevations, the ponderosa pine forest intergrades variously with chaparral, and foothill hardwood forests, of which digger pine is a common inhabitant. The conifer forest stands most closely resemble the Westside Ponderosa Pine Forest described by Holland (1986) (Appendix 6.2).

Wildlife. Table 4.5-3 provides a listing of the potential and observed wildlife species occupying the various vegetative communities likely to be impacted by the alternative.

Field sampling of the various cover types was performed by U.S. Fish and Wildlife Service and U.S. Army Corps of Engineers, in conjunction with the Habitat Evaluation Procedures (HEP) evaluation for the project. A relative rating of the value of the various cover types was made based on representative species typically occupying those cover types (U.S. Army Corps of Engineers 1991; U.S. Fish and Wildlife Service 1991). For example, species selected to represent the *North Slope Oak Woodland* included northern alligator lizard, mountain quail, black-capped chickadee, MacGillvray's warbler, western flycatcher, and gray fox; *South Slope Oak Woodland* was represented by California quail, band-tailed pigeon, western fence lizard, rufous-sided towhee, scrub jay, and desert cottontail; *Chaparral* was represented by western fence lizard, brush rabbit, California thrasher, wrentit, bobcat, and western rattlesnake; *Conifer Forest* was represented by gray fox, rufous-sided towhee, western gray squirrel, pygmy nuthatch, and western wood pewee; *Grassland* evaluation species were band-tailed pigeon, western bluebird, mourning dove, wild turkey, and western meadowlark; and Riparian habitat was represented by calliope hummingbird, willow flycatcher, dusky shrew, northern oriole, downy woodpecker, American dipper, and western screech owl.

A Habitat Suitability Index (HSI) value for each cover type was derived from the composite rating of the individual species within each cover type. The HSI were as follows:

South Slope Oak Woodland	*0.77*
North Slope Oak Woodland	*0.59*
Chaparral	*0.85*
Conifer Forest	*0.77*
Grassland	*0.73*
Riparian Shrub/Scrub	*0.80*
Gravel Bar Scrub	*0.75*

An HSI had not been formulated for the *Gravel Bar Scrub* by USFWS; however, because it was believed that the habitat value of this cover type was intermediate to grassland and riparian shrub/scrub communities, an HSI value of 0.75 was assigned to this cover type.

When multiplied by the potential number of acres of a particular cover type that would be lost as a result of project implementation, the resulting value (Habitat Units) were used to assess and compare relative losses among the different cover types.

TABLE 4.5-3 TYPICAL SPECIES OCCUPYING IN THE UPPER AMERICAN RIVER CANYONS

COMMON NAME	SCIENTIFIC NAME	COVER TYPE(1) OAK	CHA	BAR	PINE	RIP	HABITAT(3) USE	STATUS(4)
		HABITAT SUITABILITY(2)						
AMPHIBIANS AND REPTILES								
California Newt	*Taricha torosa*	S	S	-	M	O	B/F	
California Slender Salamander	*Batrachoseps attenuatus*	S	S	-	-	-	B/F	
Arboreal Salamander	*Aneides lugubris*	O	O	-	S	O	B/F	
Western Toad	*Bufo boreas*	S	M	-	M	O	B/F	
Pacific Treefrog	*Hyla regilla*	M	M	-	S	O	B/F	
Red-legged Frog	*Rana aurora*	S	S	-	-	O	B/F	CSC, FC2
Foothill Yellow-legged Frog	*Rana boylei*	S	S	-	M	O	B/F	CSC
Bullfrog*	*Rana catesbeiana*	O	O	-	M	O	B/F	
Western Pond Turtle	*Clemmys marmorata*	O	O	-	S	O	B/F	
Western Fence Lizard*	*Sceloporus occidentalis*	O	O	-	M	O	B/F	
Western Skink	*Eumeces skiltonianus*	S	O	-	S	O	B/F	
Gilbert's Skink	*Eumeces gilberti*	S	S	-	S	O	B/F	
Western Whiptail	*Cnemidophorus tigris*	S	O	-	M	S	B/F	
Southern Alligator Lizard*	*Gerrhonotus multicarinatu*	S	O	-	M	O	B/F	
Ringneck Snake	*Diadophis punctatus*	S	O	-	M	O	B/F	
Sharp-tailed Snake	*Contia tenuis*	S	S	-	S	O	B/F	
Racer	*Coluber constrictor*	S	O	-	M	S	B/F	
California Whipsnake	*Masticophis lateralis*	S	O	-	S	O	B/F	
Gopher Snake	*Pituophis melanoleucus*	O	O	-	S	O	B/F	
Common Kingsnake	*Lampropeltis getulus*	O	O	-	M	O	B/F	
Common Garter Snake	*Thamnophis sirtalis*	O	O	-	S	O	B/F	
Western Terrestrial Garter Snake	*Thamnophis elegans*	O	O	-	S	O	B/F	
Western Aquatic Garter Snake	*Thamnophis couchi*	O	O	-	S	O	B/F	
Night Snake	*Hypsiglena torquata*	S	O	-	-	M	B/F	
Western Rattlesnake	*Crotalus viridis*	O	O	-	S	O	B/F	
BIRDS								
Pied-billed Grebe	*Podilymbus podiceps*	S	S	-	S	S	F/B	
Eared Grebe	*Podiceps nigricollis*	S	S	-	S	S	F	
Western Grebe	*Aechmophorus occidentali*	M	M	-	M	M	F	
Great Blue Heron	*Ardea herodias*	S	S	-	S	S	F	

TABLE 4.5-3 TYPICAL SPECIES OCCUPYING IN THE UPPER AMERICAN RIVER CANYONS

COMMON NAME	SCIENTIFIC NAME	COVER TYPE(1)					HABITAT(3)	
		OAK	CHA	BAR	PINE	RIP	USE	STATUS(4)
		HABITAT SUITABILITY(2)						
Green-backed Heron	*Butorides striatus*	M	-	-	-	S	F/B	
Black-crowned Night-Heron	*Nycticorax nycticorax*	S	-	-	-	S	F/B	
Wood Duck	*Aix sponsa*	S	-	-	M	O	F/B	
Mallard	*Anas platyrhynchos*	S	S	-	S	S	F/B	
Common Merganser*	*Mergus merganser*	S	S	-	S	S	F/B	
Turkey Vulture*	*Cathartes aura*	S	S	-	M	M	F/B	CE, FE, CFP
Bald Eagle	*Haliaeetus leucocephalus*	M	M	-	S	M	F/B	CSC, W
Sharp-shinned Hawk	*Accipiter striatus*	S	S	-	S	S	F	CSC
Cooper's Hawk	*Accipiter cooperi*	S	M	-	S	O	F/B	
Red-tailed Hawk*	*Buteo jamaicensis*	S	M	S	M	M	F/B	CSC, CFP
Golden Eagle*	*Aquila chrysaetos*	S	S	-	M	M	F/B	
American Kestrel*	*Falco sparverius*	S	S	-	M	S	F/B	CE, FE CFP
Peregrine Falcon	*Falco peregrinus*	S	S	-	S	S	F/B	CSC
Prairie Falcon	*Falco mexicanus*	S	S	-	M	M	F/B	
California Quail*	*Lophortyx californicus*	S	O	-	M	M	F/B	
Killdeer*	*Charadrius vociferus*	M	S	S	M	-	F/B	
Spotted Sandpiper*	*Actitus macularia*	S	S	O	S	S	F/B	
Common Snipe	*Capella gallinago*	M	S	-	M	-	F/B	
Band-tailed Pigeon*	*Columba fasciata*	S	M	-	S	S	F/B	
Mourning Dove*	*Zenaida macroura*	S	S	-	S	M	F/B	
Greater Roadrunner	*Geococcyx californianus*	M	S	-		-	F/B	
Barn Owl	*Tyto alba*	O	S	-	M	-	F/B	
Western Screech Owl	*Otus asio*	O	S	-	S	O	F/B	
Great Horned Owl	*Bubo virginianus*	O	O	-	O	O	F/B	
Northern Pygmy-Owl	*Glaucidium gnoma*	O	O	-	O	O	F/B	
Burrowing Owl	*Athene cunicularia*	S	S	-	-	-	F/B	
California Spotted Owl	*Strix occidentalis*	M	-	-	M	S	F/B	CSC, FS2
Long-eared Owl	*Asio otus*	S	M	-	S	S	F/B	CSC
Common Nighthawk	*Chordeiles minor*	M	S	-	M	-	F/B	
Common Poor-Will	*Phalaenoptilus nuttallii*	S	S	-	S	-	F/B	
Black Swift	*Cypseloides niger*	M	M	-	S	S	F/B	CSC

TABLE 4.5-3 TYPICAL SPECIES OCCUPYING IN THE UPPER AMERICAN RIVER CANYONS

COMMON NAME	SCIENTIFIC NAME	COVER TYPE(1)					HABITAT(3)	
		OAK	CHA	BAR	PINE	RIP	USE	STATUS(4)
		HABITAT SUITABILITY(2)						
Vaux's Swift	*Chaetura vauxi*	M	M	-	M	M	F/B	
White-throated Swift	*Aeronautes sacatalis*	S	S	-	S	S	F/B	
Black-chinned Hummingbird	*Archilochus alexandri*	S	M	-	S	S	F/B	
Anna's Hummingbird*	*Calypte anna*	S	S	-	S	S	F/B	
Belted Kingfisher*	*Megaceryle alcyon*	S	M	-	O	S	F/B	
Acorn Woodpecker*	*Melanerpes formicivorous*	S	-	-	M	M	F/B	
Nuttall's Woodpecker	*Picoides nuttallii*	S	M	-	-	S	F/B	
Downy Woodpecker	*Picoides pubescens*	M	-	-	M	O	F/B	
Western Wood Pewee	*Contopus sordidulus*	S	-	-	S	S	F/B	
Willow Flycatcher	*Empidonax traillii*	-	-	-	-	O	F/B	CSC, FC2
Hammond's Flycatcher	*Empidonax hammondii*	M	M	-	S	-	F	
Pacific-slope Flycatcher*	*Empidonax difficilis*	M	M	-	S	-	F/B	
Black Phoebe*	*Sayornis nigricans*	S	S	-	S	O	F/B	
Say's Phoebe	*Sayornis saya*	S	O	-	S	-	F/B	
Ash-throated Flycatcher	*Myiarchus cinerascens*	S	O	-	S	S	F/B	
Western Kingbird*	*Tyrannus verticalis*	S	S	-	M	-	F/B	
Tree Swallow	*Iridoprocne bicolor*	S	S	-	S	O	F/B	
Violet-green Swallow*	*Tachycineta thalassina*	S	M	-	S	O	F/B	
Northern Rough-winged Swallow	*Stelgidopteryx ruficollis*	O	O	-	S	O	F/B	
Cliff Swallow*	*Petrochelidon pyrrhonota*	O	O	-	M	M	F/B	
Barn Swallow	*Hirundo rustica*	O	O	-	S	S	F/B	
Scrub Jay*	*Aphelocoma coerulescens*	S	M	-	M	S	F/B	
American Crow	*Corvus brachyrhynchos*	M	M	-	M	S	F/B	
Common Raven	*Corvus corax*	O	O	-	S	S	F/B	
Plain Titmouse*	*Parus inornatus*	S	D	-	M	S	F/B	
Bushtit	*Psaltriparus minimus*	S	M	-	M	M	F/B	
White-breasted Nuthatch	*Sitta carolinensis*	O	-	-	S	M	F/B	
Canyon Wren	*Catherpes mexicanus*	S	S	-	S	M	F/B	
Bewick's Wren*	*Thryomanes bewickii*	S	O	-	M	M	F/B	
American Dipper*	*Cinclus mexicanus*	-	-	-	-	O	F/B	
Ruby--crowned Kinglet*	*Regulus calendula*	O	O	-	M	S	F/B	

TABLE 4.5-3 TYPICAL SPECIES OCCUPYING IN THE UPPER AMERICAN RIVER CANYONS

COMMON NAME	SCIENTIFIC NAME	COVER TYPE(1) OAK	CHA	BAR	PINE	RIP	HABITAT(3) USE	STATUS(4)
		HABITAT SUITABILITY(2)						
Blue-gray Gnatcatcher	*Polioptila caerulea*	S	M	-	M	M	F/B	
Western Bluebird*	*Sialia mexicana*	M	-	-	-	-	F/B	
Swainson's Thrush	*Catharus ustulata*	M	-	-	M	S	F/B	
Hermit Thrush*	*Catharus guttatus*	S	S	-	M	O	F/B	
American Robin*	*Turdus migratorius*	S	S	-	S	O	F/B	
Wrentit*	*Chamaea fasciata*	S	O	-	M	-	F/B	
Northern Flicker*	*Colaptes auratus*	S	M	-	S	S	F/B	
California Thrasher	*Toxostoma redivivum*	S	O	-	-	-	F/B	
Phainopepla	*Phainopepla nitens*	O	S	-	-	M	F/B	
Loggerhead Strike	*Lanius ludovicianus*	S	M	-	-	M	F/B	
European Starling	*Sturnus vulgaris*	O	O	-	-	S	F/B	
Warbling Vireo	*Vireo gilvus*	M	-	-	S	O	F/B	
Orange-crowned Warbler	*Vermivora celata*	S	O	-	S	O	F/B	
Yellow Warbler	*Denroica petechia*	O	S	-	S	O	F/B	CSC
Black-throated Gray Warbler	*Dendroica nigrescens*	S	S	-	S	M	F/B	
Common Yellowthroat	*Geothlypis trichas*	S	S	-	S	O	F/B	
Yellow-breasted Chat	*Icteria virens*	-	-	-	-	S	F/B	CSC
Black-headed Grosbeak*	*Fheucticus melanocephalu*	S	M	-	M	O	F/B	
Lazuli Bunting*	*Passerina amoena*	S	S	-	S	S	F/B	
Rufous-side Towhee*	*Pipilo erythrophthalmus*	S	O	-	M	S	F/B	
California Towhee*	*Pipilo fuscus*	S	O	-	-	S	F/B	
Chipping Sparrow*	*Spizella passerina*	S	S	-	S	-	F/B	
Lark Sparrow*	*Chondestes grammacus*	S	O	-	M	-	F/B	
Sage Sparrow*	*Amphispiza belli*	-	O	-	-	-	F/B	
Song Sparrow*	*Melospiza melodia*	M	-	-	M	O	F/B	
Golden-crowned Sparrow*	*Zonotrichia atricapilla*	S	S	-	M	M	F	
Dark-eyed Junco*	*Junco hyemalis*	S	M	-	S	O	F/B	
Tricolored Blackbird	*Agelaius tricolor*	M	-	-	-	S	F/B	FC2
Brewer's Blackbird	*Euphagus cyanocephalus*	S	S	-	M	S	F/B	
Brown-headed Cowbird	*Molothrus ater*	S	M	-	M	O	F/B	
Northern Oriole	*Icterus galbula*	S	M	-	M	O	F/B	

TABLE 4.5-3 TYPICAL SPECIES OCCUPYING IN THE UPPER AMERICAN RIVER CANYONS

COMMON NAME	SCIENTIFIC NAME	COVER TYPE(1)					HABITAT(3)	
		OAK	CHA	BAR	PINE	RIP	USE	STATUS(4)
		HABITAT SUITABILITY(2)						
House Finch	*Carpodacus mexicanus*	S	S	-	-	S	F/B	
Lesser Goldfinch	*Carduelis psaltria*	O	S	-	S	S	F/B	
American Goldfinch*	*Carduelis tristis*	S	S	-	-	O	F/B	
MAMMALS								
Virginia Opossum	*Didelphis virginiana*	M	M	-	S	S	F/B	
Ornate Shrew	*Sorex ornatus*	M	M	-	M	S	F/B	
Broad-footed Mole	*Scapanus latimanus*	M	M	-	M	S	F/B	
Yuma Myotis	*Myotis humanensis*	S	M	-	S	S	F/B	
California Myotis	*Myotis californicus*	S	S	-	M	M	F/B	
Western Pipistrelle	*Pipistrellus hesperus*	O	M	-	M	M	F/B	
Big Brown Bat	*Eptesicus fuscus*	S	M	-	M	M	F/B	
Red Bat	*Lasiurus borealis*	O	S	-	S	O	F/B	
Hoary Bat	*Lasiurus cinereus*	S	M	-	S	M	F/B	
Brazilian Free-tailed Bat	*Tadarida brasiliensis*	M	S	-	S	M	F/B	
Brush Rabbit	*Sylvilagus bachmani*	S	O	-	-	-	F/B	
Desert Cottontail	*Sylvilagus audubonii*	S	O	-	-	-	F/B	
Black-tailed Hare	*Lepus californicus*	S	O	-	M	-	F/B	
Long-eared Chipmunk	*Eutamias quadrimaculatus*	-	-	-	S	M	F/B	
California Ground Squirrel	*Spermophilus beecheyi*	S	S	-	S	M	F/B	
Western Gray Squirrel	*Sciurus griseus*	S	M	-	S	M	F/B	
California Pocket Mouse	*Perognathus californicus*	M	S	-	-	-	F/B	
California Kangaroo Rat	*Dipodomys californicus*	S	S	-	-	-	F/B	
Beaver	*Castor canadensis*	-	-	-	-	O	F/B	
Deer Mouse	*Peromyscus maniculatus*	S	S	-	S	S	F/B	
Brush Mouse	*Peromyscus boylii*	S	S	-	S	-	F/B	
Pinyon Mouse	*Peromyscus truei*	S	O	-	S	S	F/B	
Dusky-footed Woodrat	*Neotoma fuscipes*	S	S	-	S	S	F/B	
Muskrat	*Ondatra zibethicus*	-	-	-	-	O	F/B	
Coyote	*Canis latrans*	S	O	-	S	M	F/B	
Gray Fox*	*Urocyon cinereoargenteus*	S	O	-	S	M	F/B	
Ringtail	*Bassariscus astutus*	S	S	-	M	O	F/B	

TABLE 4.5-3 TYPICAL SPECIES OCCUPYING IN THE UPPER AMERICAN RIVER CANYONS

COMMON NAME	SCIENTIFIC NAME	COVER TYPE(1)					HABITAT(3)	
		OAK	CHA	BAR	PINE	RIP	USE	STATUS(4)
		HABITAT SUITABILITY(2)						
Raccoon	*Procyon lotor*	M	M	-	M	O	F/B	
Long-tailed Weasel	*Mustela frenata*	M	S	-	S	O	F/B	
Mink	*Mustela vison*	-	-	-	-	O	F/B	
Badger	*Taxidea taxus*	S	S	-	S	S	F/B	CSC
Western Spotted Skunk	*Spilogale gracilis*	S	S	-	S	M	F/B	
Striped Skunk	*Mephitis mephitis*	S	S	-	S	O	F/B	
Mountain Lion	*Felis concolor*	M	S	-	S	-	F/B	CSC, FC2
Bobcat	*Felis rufus*	S	S	-	S	M	F/B	
Mule Deer*	*Odocoileus hemionus*	S	S	-	S	S	F/B	

* Observed during field surveys.

(1) Cover Type Code
- OAK = Oak Woodland
- CHA = Chaparral
- BAR = Gravel Bar Scrub
- PINE = Conifer Forest
- RIP = Riparian Shrub/Scrub

(2) Habitat Suitability
- O = Optimal
- S = Suitable
- M = Marginal

(3) Habitat Use:
- B = Breeding
- F = Foraging

(4) Status
- FE = Listed as Endangered by Federal Government
- FT = Listed as Threatened by Federal Government
- CE = Listed as Endangered by the State of California
- CT = Listed as Threatened by State of California
- CFP = California Department of Fish and Game "Fully Protected" species
- CSC = California Deparmtent of Fish and Game Species of Special Concern
- FC2 = Federal Category 2: Species may warrant listing, but sufficient biological information to support a proposal rule is lacking
- BL = Blue List: Avian Species of Special Concern as identified in American Birds.

Sources: Brown et al. (1986); Clark and Wheeler (1987); Jameson and Peeters (1986); Peterson (1990); Stebbins (1966); Verner and Bos (1980); Zeiner et al. (1988, 1990a, 1990b).

Fish and Aquatic Resources. The reach between Cherokee and Mammoth Bars is characterized by a series of riffles, pools and runs. The California Department of Fish and Game identified approximately 14 pools/mile below Brushy Canyon (located approximately 21.5 miles upstream of Cherokee Bar on the Middle Fork of the Middle Fork at elevation 1,720 feet, msl) and, approximately 26 pools/mile above this point (Gerstung 1969). Visual estimates conducted by the USFWS in 1989 found that the average riffle area was 132 feet long, 106 feet wide, and 6 feet deep. The average pool was found to be approximately 353 feet long, 100 feet wide and 16 feet deep (USFWS 1991). The substrate was found to be predominantly gravel and cobble with smaller proportions of boulders, sand, and fine sediments.

Several previous studies were reviewed to determine the past and present composition of the fishery of the North and Middle Forks of the American River (U.S. Fish and Wildlife Service 1991; Gerstung 1989; Harvey 1986; Harvey et al. 1982; U.S. Department of Agriculture 1978; California Department of Fish and Game 1934, 1938, and 1977; Gerstung 1971; California State Water Resources Board 1955; Moffett et al. 1948; and Sumner and Smith 1942). Prior to European settlement of California, the endemic fish fauna included chinook salmon, rainbow trout, and steelhead (sea-run rainbow trout), Sacramento squawfish, Sacramento sucker, hardhead, riffle sculpin, western roach, speckled dace, and Pacific lamprey (Table 4.5-4). Construction of downstream barriers eliminated white sturgeon, and the anadromous species (e.g., steelhead, chinook salmon, and Pacific lamprey). Since European settlement, several non-native species have been introduced into the North and Middle Forks, principal of which are smallmouth bass, eastern brook trout, brown trout, kokanee salmon (non-anadromous sockeye salmon), and green sunfish.

Prior to the construction of the Placer County Water Agency's hydroelectric project in the late 1960s, the Middle Fork's warm stream temperature limited the production of coldwater species, such as brown and rainbow trout, to areas above its confluence with the Rubicon River. However, discharges from the cold water pools from Hell Hole and French Meadows reservoirs through the Ralston Afterbay have extended the range of trout to the vicinity of Ruck-A-Chucky rapids (U.S. Fish and Wildlife Service 1991; Gerstung 1989). Below Ruck-A-Chucky, in the vicinity of the proposed dredging project, stream temperatures limit the production of trout and favor warm water species such as smallmouth bass (Gerstung 1989). Similar findings were reported by the U.S. Fish and Wildlife Service (1991) based on sampling performed in 1989.

Typical instream macroinvertebrates found in the American River watershed and similar western Sierra Nevada streams include aquatic beetles (*Elmidae*), stoneflies (*Calineuria sp., Chloroperlidae, Capniidae*), mayflies (*Baetis sp. Tricorythodes sp.*), caddisflies (*Hydropsyche sp.*), and aquatic diptera (*Chironomidae, Simuliiidae*) (Martin and Knight 1989; Harvey 1986; and California Department of Fish and Game 1938).

TABLE 4.5-4 CURRENT AND HISTORIC COMPOSITION OF FISHERY ON NORTH AND MIDDLE FORKS OF THE AMERICAN RIVER

Common Name	Scientific Name	Endemism	Occupancy	Breeding Habitat	Temperature	Forage Preference
FISHES						
Pacific lamprey	*Lampetra tridentata*	E(1)	H(2)	gravel bottom	<80	parasite on fish
Chinook salmon	*Oncorhynchus tshawytscha*	E	H	gravel riffles	50-55	drift; opportunistic
Kokanee salmon	*Oncorhynchus nerka*	I	P	gravel riffles	45-55	zooplankton
Rainbow trout	*Oncorhynchus mykiss*	E	P	gravel riffles	50-58	macroinverts; drift and bottom
Brown trout	*Salmo trutta*	I	P	gravel riffles	55-60	bottom feeder; drift
Eastern brook trout	*Salvelinus fontinalis*	I	P	gravel riffles	50-55	drift
Tui chub	*Gila bicolor*	E	P	vegetated beds	55-60 (S)	omnivorous
Thicktail chub	*Gila crassicauda*	E	*			small fish; invertebrates
Hitch	*Lavinia exilicauda*	E	P	sand/gravel bottoms	55-65 (S)	omnivorous
California roach	*Hesperoleucus symmetricus*	E	P	rocky bottoms	<90	algae
Hardhead	*Mylopharodon conocephalus*	E	P	gravel riffles		bottom; invertebrates and plants
Sacramento squawfis	*Ptychocheilus grandis*	E	P	rocky bottoms	60-70	fish ;invertebrates
Spreckled dace	*Rhinichthys osculus*	E	P	shallow gravels	<90	bottom inverebrates
Sacramento sucker	*Catostomus occidentalis*	E	P	gravel riffles	42-52 (S)	bottom feeder; omnivorous
Green sunfish	*Lepomis cyanellus*	I	P	gravel bottom	60-70	opportunistic predator
Smallmouth bass	*Micropterus dolomieui*	I	P	gravel/sand bottom	60-70	fish, amphibians, insects
Riffle sculpin	*Cottus gulosus*	E	P	under rocks in riffles		bottom; carnivorous

CODES:

(1) E = Endemic
I = Introduced

(2) H = Historic Inhabitant
P = Present Inhabitant
* = Extinct

(3) S = Preferred spawning temperature

SOURCES:

Bell (1986)
California Department of Fish and Game (1977, 1938, 1934)
California State Water Resurces Board (1955)
Gerstung (1971)
Gerstung (1989)
Harvey et al. (1982)
Harvey (1986)
McGinnis (1984)
Moffett et al. (1948)
Moyle (1976)
Moyle et al. (1989)
Sumner and Smith (1942)
U.S. Department of Agriculture (1978)
U.S. Fish and Wildlife Service (1991)

Sensitive Species. The Federal Endangered Species Act of 1973 (50 CFR 17) provides legal protection, and requires definition of critical habitat and development of recovery plans for plant and animal species in danger of extinction. California has a parallel mandate embodied in the California Endangered Species Act of 1984 and the California Native Plant Protection Act of 1977. These laws regulate the listing of plant and animal species as endangered, threatened, or in the case of plants, rare. In addition, the Federal Endangered Species Act requires Federal agencies to make a finding on all Federal actions, including the approval by an agency of a public or private action, such as the issuance of a Section 10/404 permit, as to the potential to jeopardize the continued existence of any listed species potentially impacted by the action. Species listed by the State are not necessarily protected by the Federal protection statutes. Under the State laws, the California Department of Fish and Game is empowered to review projects for their potential impacts to listed species and their habitats.

In addition to formal endangered and threatened listings by Federal and State governments are the listing of species of special interest due to their limited distribution, declining populations, diminishing habitat, or unusual scientific, recreational, or educational value. These species are not afforded the same legal protection as listed species, but may be added to official lists in the future. There are two general categories of special interest species:

1) *those species that are candidates for official federal or state listing as threatened or endangered;*

2) *those species which are not candidates, but which have been unofficially identified as a species of special interest by private conservation organizations or local government agencies.*

Federal candidate species are assigned to one of two categories depending on the current state of knowledge of the species and its biological appropriateness for listing. Federal Category 1 candidate species (FC1) include taxa for which the USFWS currently has compiled substantial information on biological vulnerability and threats to support the appropriateness of proposing to list the taxa as endangered or threatened species. Federal Category 2 candidates (FC2) includes taxa for which sufficient information is available to indicate possible listings, but for which additional data are required on vulnerability and threats. The state also maintains lists for Candidate-Endangered Species (SCE) and State Candidate-Threatened Species (SCT).

A list of endangered, threatened, and candidate species potentially inhabiting the project area was compiled from reports prepared by the U.S. Fish and Wildlife Service and California Department of Fish and Game reports (U.S. Army Corps of Engineers 1991), and from retrievals from the RareFind database for the Auburn, Greenwood, Auburn, Brown's Ferry, and Lake Combie topographic quadrangles. These data are listed in Table 4.5-5.

TABLE 4.5-5 SENSITIVE SPECIES POTENTIALLY OCCURRING IN THE PROJECT AREA[1]

COMMON NAME	SCIENTIFIC NAME	STATUS	TYPICAL HABITAT
VEGETATION			
El Dorado morning glory	Calystegia stebbinsii	SE, FC2	Gabbroic Northern Mixed Chaparral
Pine Hill ceanothus	Ceanothus roderickii	SR, FC2	Gabrroic Northern Mixed Chaparral
Red Hills soaproot	Chlorogalum grandflorum	FC2	Dry, rocky, open serpentine soils
Pine Hill flannelbush	Fremontodendron decumbens	SR, FC2	Gabbroic Northern Mixed Chaparral
El Dorado bedstraw	Galium californicum sierrae	SR, FC2	Gabbroic Northern Mixed Chaparral
Bisbee Peak rush-rose	Helianthemum suffrutescens	FC2	Chaparral
Stebbins' phacelia	Phacelia stebbinsii	FC2	Conifer forest, streamside
Layne's butterweed	Senecio layneae	SR, FC2	Gabbroic Northern Mixed Chaparral
El Dorado County mule ears	Wyethia reticulata	FC2	Gabbroic Northern Mixed Chaparral
INSECTS			
Spiny rhyacophilian caddisfly	Rhyacophila spinata	FC2	Swift, Sierra Nevada streams
Valley elderberry longhorn beetle	Desmocerus dimorphus californicus	FT	Elderberry plants
Darlington's ground beetle	Nebria darlingtoni	2R	
AMPHIBIANS AND REPTILES			
California red-legged frog	Rana aurora draytoni	FC2	Various habitats with rocky streams
BIRDS			
Cooper's hawk	Accipiter cooperii	CSC, CFP	deciduous trees in riparian canyons
American peregrine falcon	Falco peregrinus anatum	SE,FE	Various habitats, including rivers
Bald eagle	Haliaeetus leucocephalus	SE, FE	Lakes, free-flowing rivers
Tricolored blackbird	Agelaius tricolor	FC2	Emergent marshes, willow thickets
California spotted owl	Strix occidentalis	FC2	Oak and oak-conifer stands

[1]Based on coordination with U.S. Fish and Wildlife Service and California Department of Fish and Game (U.S. Army Corps of Engineers, 1991), and RareFind database retrievals.

For the Middle Fork Sand and Gravel alternative, the following species have been identified as potentially inhabiting the project area:

Bald Eagle. Bald eagles have not nested in the American River watershed area in recent years; however, sightings of wintering bald eagles around Folsom Reservoir are fairly common. Occasionally, an eagle is observed along the lower American River. Bald eagles are observed less frequently above Folsom Reservoir, and ground and aerial surveys conducted in winter/spring 1989-90 did not detect any wintering eagles between the lower American River and the upper watershed (approximate elevation 1,200 feet, msl).

American Peregrine Falcon. The peregrine falcon is known to nest and overwinter in the western Sierra Nevada. Typical nesting habitat includes nest on a ledge of large cliff faces; however, the use of tall trees and buildings has been recorded. Nesting and wintering habitats are variable and include wetlands, woodlands, other forested habitat, cities, agricultural areas, and coastal habitats (CDFG 1990).

Valley Elderberry Longhorn Beetle. Clumps of elderberry plants were found on the gravel bars along the Middle Fork. These plants are the host plant for the valley elderberry longhorn beetle (VELB), a federally listed Threatened species. Examination of the plants for signs of the VELB (adults, frass, emergence holes) was conducted by the U.S. Fish and Wildlife Service (1991) and during field reconnaissance surveys in July of 1991. In each survey, signs of VELB inhabitation were not found; however, presence of suitable habitat offered by the plants should presume existence of the beetle (U.S. Fish and Wildlife Service 1991).

Tricolored Blackbird. No reports of nesting colonies of tricolored blackbirds have been recorded in Placer or El Dorado Counties. Suitable breeding habitat for the tricolored blackbird includes emergent wetland vegetation (dense cattails or tules) or willow, wild rose, and blackberry thickets near fresh water. Feeding habitat includes agricultural fields and grasslands. The project site offers limited habitat value based on the absence of preferred foraging habitat (U.S. Fish and Wildlife Service 1991).

California Spotted Owl. The California subspecies of the spotted owl (*Strix occidentalis occidentalis*) occurs in the Sierra Nevada range and mountainous areas of Southern California. It is currently listed as a federal candidate species, in contrast to the northern subspecies (*S. o. caurina*), which is a federally listed endangered species (U.S. Fish and Wildlife Service 1991). Studies have documented the downslope migration to winter ranges as low as 885 feet in the Placerville/Auburn area (U.S. Fish and Wildlife Service 1991).

California Red-legged Frog. The red-legged frog requires quiet, permanent pools of streams and marshes with extensive vegetation for escape cover. Potential habitat in the project area include backwater areas and isolated permanent and seasonal ponds with emergent vegetation, isolated ponds, and canals and drainages that lack bullfrogs or other large aquatic predators (U.S. Fish and Wildlife Service 1991).

Gabbroic Northern Mixed Chaparral Endemics. The species associated with this community include Pine Hill ceanothus, Stebbins' morning glory, Layne's butterweed, El Dorado mule ears, Red Hills soaproot, Pine Hill flannelbush, and El Dorado bedstraw, and are commonly known as the "Pine Hill endemics." Vegetative reconnaissance surveys conducted in 1989 and 1991 failed to reveal the presence of any of the Gabbroic Northern Mixed Chaparral community along the Middle and North Forks of the American River in the project area. Further, none of the "Pine Hill endemics" associated with the gabbro chaparral were detected. Finally, a review of soil and geologic maps (U.S. Department of Agriculture 1974, 1979; Youngs 1988) did not reveal the presence of gabbro or serpentine soils or extrusions in the vicinity of the project area.

Bisbee Peak Rush-rose. The Bisbee Peak rush-rose is an inhabitant of open, dry chaparral and oak-pine woodland, on olivine schist, gabbro, or serpentine soils. It has not been recorded in the project area, nor was the species observed during reconnaissance surveys. The northernmost report of the species is within the Pilot Hill area of El Dorado County (Smith and Berg 1988).

Stebbins' Phacelia. Stebbins' phacelia has not been reported in the project area, nor found during vegetative surveys. It has been recorded in El Dorado and Placer Counties, but at higher elevations (3,000-4,800 feet) within the lower montane coniferous forest (Smith and Berg 1988).

Spiny Rhyacophilan Caddisfly. The spiny rhyacophilan caddisfly has been reported from Placer County in Ladys Canyon of the Middle Fork American River watershed. The species occupies cool, running water and is presumed extant in the vicinity of the project.

Old Cool Quarry (Spreckles)

Vegetative Cover Types. The area from which aggregate would be quarried at the Old Cool Quarry (Figure 4.5-3) has been previously cleared of vegetation and is presently exposed substrate. No additional vegetation clearing would be necessary (L. Bartley, pers. comm., 1991).

Wildlife. The quarry does not contain vegetative cover necessary to support wildlife populations.

Fish and Aquatic Resources. No fish or aquatic resources are located in the quarry area.

Sensitive Species

Yate's snail. The Yate's snail, or Tight Coin (*Ammonitela yatesi*), a federal candidate species, inhabits limestone caves and outcroppings. It has been reported by the RareFind database in the vicinity of the Old Cool Quarry.

Cool Quarry Amphibolite

Vegetative Cover Types. The proposed Cool Amphibolite Quarry site is located immediately west of the existing Old Cool Quarry on a north-facing slope along the south slope of the Middle Fork American River canyon (Figure 4.5-3). The site is approximately 40 acres, and composed principally of *North Slope Oak Woodland* (15.8 acres), *South Slope Oak Woodland* (3.9 acres), *Conifer Forest* (11.9 acres), and *Riparian Shrub/Scrub* (7.9 acres). A more detailed description of thee cover types is included in the Middle Fork section.

Wildlife. Table 4.5-3 lists the typical species associated with *Oak Woodlands, Conifer Forest*, and *Riparian Shrub/Scrub* vegetative communities found on the proposed quarry site.

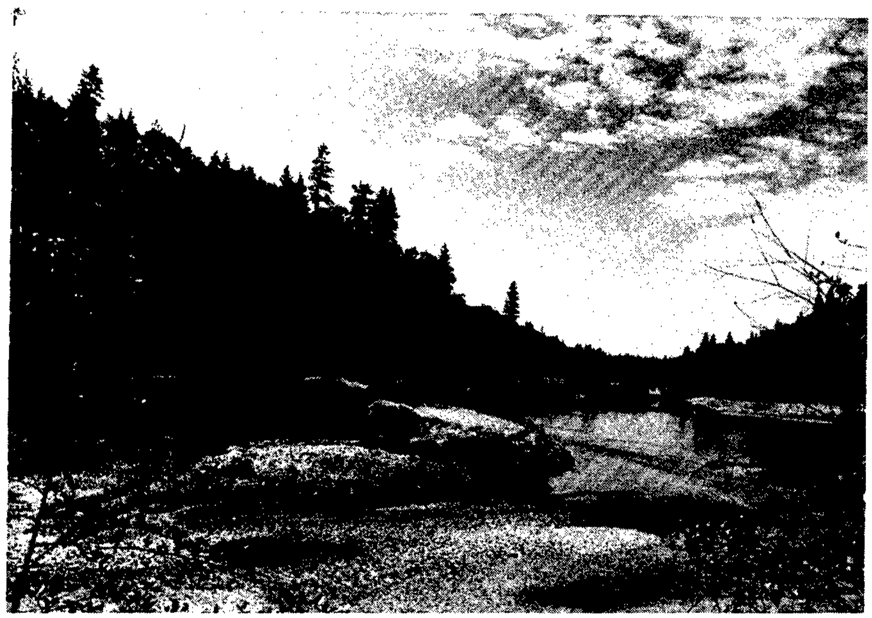
Fish and Aquatic Resources. No fish or aquatic resources are located in the quarry area.

Sensitive Species. No sensitive species have been identified in the vicinity of the Cool Quarry Amphibolite. However, potential habitat for bald eagles (roosts), peregrine falcons (nesting and wintering), and the Yate's snail may be found on site.

Bear River and Chevreaux Quarry

Vegetative Cover Types. The Joe Chevreaux Company Quarry located on the upstream end of Lake Combie on the Bear River at the Placer-Nevada County line mines aggregate from the lake and river and also from hardrock quarries adjacent to the river. The active portion of the rock quarry is approximately 40 acres; however, over 1,000 acres in Placer and Nevada counties are being reserved for quarry operation by the company (California Department of Conservation 1991). Based on ground and aerial photographs of the site, the predominant cover type is mixed oak-conifer woodlands, similar in composition to the *South Slope Oak Woodland* community described above (Figure 4.5-5). Conservative estimates of the volume of quarriable material, based on a 10-foot mining depth and a 700-acre quarry site, was estimated by the Division of Mines and Geology to be approximately 20 million cubic yards (Dupras 1983).

Wildlife. Typical wildlife species associated with the Oak Woodlands are described in Table 4.5-3.



View of Chevreaux Sand and Gravel Operation at Bear River/Lake Combie. Oak woodlands, as shown on slope at left of photograph is typical of vegetative community on and near hardrock quarry.

Biological Resources - Chevreaux Site

FIGURE 4.5-5

Fish and Aquatic Resources. No fish or aquatic resources are associated with the hardrock quarry site.

Sensitive Species. No sensitive species have been identified in the project area.

Mississippi Bar Sand and Gravel Deposits

Vegetative Cover Types. The Mississippi Bar site has been characterized by the U.S. Bureau of Reclamation (1988) as an area of sparely vegetated dredger tailings. Typically, the tailing piles are vegetated with grasses and ruderal vegetation, such as yellow star thistle, mustard, vetch, and an occasional oak tree. The swale areas, or slickens, between the tailing piles are often more moist and support riparian and foothill woodland vegetation composed of Fremont cottonwood, interior live oak, blue oak, valley oak, digger pine, and willow. Typical understory vegetation includes coyote bush, elderberry, poison oak, and blackberry. Analysis of aerial photography of the site (scale = 1:3,600) taken on March 8, 1991 indicated that approximately 30 acres of the 160 acre site are vegetated in *Riparian Shrub-Scrub*, 124 acres in exposed tailings, and 9.3 acres in open water, including a 7.5 acre wetland.

Wildlife. Typical species reported on the site by the USBR (1988) include blacktail deer, raccoon, opossum, California ground squirrel, gray fox, scrub jay, red-tailed hawk, acorn woodpecker, California quail, western fence lizard, garter snake, and the western rattlesnake. A great blue heron rookery and a wetland preserve occur in the vicinity of the project site.

Fish and Aquatic Resources. No current information exists concerning the composition and abundance of fish and aquatic resources inhabiting the gravel pits on site. However, it is likely that only warm water species, such as bluegill, green sunfish, crappie, mosquitofish, and various cyprinids could tolerate the temperature and turbidity normally associated with active sediment ponds.

Sensitive Species. Mississippi Bar was inspected for the presence of two species: the Valley Elderberry Longhorn Beetle and Sanford's sagittaria (*Sagittaria sanfordii*), federal candidate species. Surveys failed to reveal the presence of Sanford's sagittaria; however, over 75 elderberry plants were found on-site. Although no VELB specimens were observed, over 25 percent of the elderberry plants were found to have emergence holes, which strongly indicates the presence of the VELB in the vegetation (U.S. Bureau of Reclamation 1988).

The RareFind database (California Department of Fish and Game 1991) reported the presence of Cooper's hawk, a California Species of Special Concern and Fully Protected Species, nesting on Mississippi Bar.

Yuba River Dredge Fields

Vegetative Cover Types. The predominant cover at the gravel mining site is exposed rock, gravel, and sand. Occasional stands of riparian vegetation can be observed in the moist swales between the tailings. Analysis of aerial photographs of a typical operation (Western Aggregates), indicated the presence of approximately 7 acres of scattered woodlands on a 111 acre tailing area. The remaining area was exposed sand and gravel and gravel ponds.

Wildlife. Typical wildlife associated with the *Riparian Shrub/Scrub* community are described in Table 4.5-3.

Fish and Aquatic Resources. No current information exists concerning the composition and abundance of fish and aquatic resources inhabiting the gravel pits in the Yuba Dredge Fields. However, it is likely that only warm water species, such as bluegill, green sunfish, crappie, mosquitofish, and various cyprinids could tolerate the temperature and turbidity normally associated with active sediment ponds.

Sensitive Species. No federal or state-listed endangered or threatened species have been reported from the Yuba Gold Dredge Fields. However, the tricolored blackbird, a federal candidate species, has historically used the area for nesting.

4.5.2 IMPACT ANALYSIS

Thresholds of Significance. Thresholds of Significance were identified from the California Environmental Quality Act (California Office of Planning and Research 1988) and local/regional plans and ordinances. Using these guidelines, the proposed project was evaluated to determine if significant impacts to biological resources would result from project implementation. Significance thresholds were based on the following:

A. Conflict with adopted environmental plans and goals in the community where it is located (CEQA Guidelines, Appendix G[a]).

B. Substantially affect a rare or endangered species (CEQA Guidelines Appendix G[c]);

C. Interfere substantially with the movement of any resident or migratory fish or wildlife species (CEQA Guidelines Appendix G[d]);

D. Substantially diminish habitat for fish, wildlife, or plants (CEQA Guidelines Appendix G[t]);

E. Involve the use, production or disposal of material which pose a hazard to animal or plant populations in the area affected (CEQA Guidelines Appendix G[v]);

F. Adversely impact a plant or animal taxa considered locally important, based on the following criteria:

1. *Taxa (species, subspecies, or varieties) that are limited in distribution in the county or region, or are endemic (limited to a specific area) to the region;*

2. *Taxa that are at the extremes of their range or are disjunct from the known range for the taxon;*

3. *Taxa whose habitat requirements make them susceptible to local extinctions as a consequence of development, the introduction of barriers to movement, and/or accompanying increases in human activity;*

4. *Populations of particular species which exhibit unusual adaptations or are quality examples of the species; and*

5. *Taxa which are considered sensitive by recognized monitoring groups (i.e., Audubon Society, California Native Plant Society, etc.).*

G. Impact a community considered locally important based on the following criteria:

1. *Plant communities of habitat types that are of singular or limited occurrence within the county or project area;*

2. *Plant communities or habitat types that are critical or essential habitat for rare, threatened, endangered or locally important species;*

3. *Plant communities, habitat types, or geographic areas which link substantial, intact open space areas;*

4. *Plant communities or habitat types that exhibit characteristics approximating pristine conditions;*

5. *Type localities for particular species of plants or animals;*

6. *Communities considered sensitive by recognized monitoring groups such as the California Natural Diversity Data Base, California Department*

of Fish and Game, Audubon Society, California Native Plant Society; and,

7. *Ephemeral or perennial wetlands defined as areas which sporadically, seasonally or perennially serve to emit, conduct, or impound water, making it available to water-dependent and/or facultative associations of plants or animals.*

Project impacts are further classified with respect to the ability to off-set, avoid, or mitigate the impact. The classification system is as follows:

Class I — *Significant adverse impacts which cannot be mitigated or avoided. A significant unmitigable impact is a problem for which a solution has not been formulated due either to the limits of technical and/or scientific knowledge or infeasibility from technical, economic, and/or political basis.*

Class II — *Significant adverse environmental impacts that can be feasibly mitigated or avoided. In these cases, the consequences of a project are considered sufficiently serious that some form of mitigation planning is needed. These mitigations can involve modifications to the project, changing the project design to avoid conflicts with environmental values, or performing data collection procedures prior to construction. Under section 15091 of CEQA, decision-makers are required to make findings that impacts have been mitigated as completely as possible to approve a project with Class II impacts.*

Class III — *Adverse project impacts found not to be significant. Adverse impacts describe the consequences of a project that are not sufficiently disruptive to require mitigation measures. Minor changes in the environment that have no serious consequences on the abundance or diversity of plant or animal life, for example, are classified as adverse but not significant.*

Class IV — *Beneficial project impacts.*

Middle Fork Sand and Gravel Deposits

Impacts on Vegetation. Acquisition of sand and gravel from the Middle Fork bars would be accomplished on the bars and not directly in the river. Pits would be excavated to depths of 30 feet and below the water table. The wet pits would be separated from the river by a series of gravel levees. Because of the extent of the volume required, all vegetation on the impacted bars would be removed. Based on estimated areal coverage of the principal vegetative communities described above, approximately 34 acres of riparian shrub/scrub and 75 acres of gravel bar scrub would be removed (Table 4.5-6).

Minshall et al. (1989) observed that, although vegetation generally covers a small proportion of a gravel bar, the species diversity is often very high in comparison to other riparian habitats. This is due, in part, to the frequent disturbance and coarse substrate which have equal potential for the initial establishment of both upland and riparian species. Riparian vegetation performs several functions important to the river ecosystem. The riparian vegetation provides sources of nutrients through allochthonous detrital inputs, e.g., dead leaves, twigs, frass, insect drop, dissolved organic matter, etc. (Knight and Bottorff 1984). These coarse nutrient sources are, in turn, broken down by aquatic macroinvertebrates, such as stonefly nymphs, cranefly larvae, and caddisfly larvae, into fine particulate matter available to other aquatic organisms (mayfly larvae, midge larvae, and blackfly larvae), and downstream export. Riparian vegetation provides overhead shading important in the maintenance of stream temperatures, and provides nesting, perching, roosting, and foraging habitat for birds and cover for arboreal mammals.

In terms of scarcity, riparian habitats occupy less than 5 percent of their historic range in California. Smith (1977) estimated that riparian vegetation occupied over 775,000 acres in 1848; however, by 1977, the acreage declined to 12,000 acres. On this basis, riparian habitats are considered rare by the California Department of Fish and Game (Holland 1986). None of the upland cover types described above are considered rare by the California Department of Fish and Game (Holland 1986). Further, these upland cover types have not experienced as substantial a loss as the riparian cover types. Barbour and Major (1988) reported that oak woodland account for over 9.5 percent of the California landscape (9.5 million acres), while chaparral comprises 8.5 percent (8.5 million acres), and mixed conifer forests account for 13 percent (13 million acres).

In terms of relative habitat value, of the 161 species potentially inhabiting various cover types in the project area for foraging and/or breeding (Table 4.5-3), approximately 136 (84 percent) use the riparian community, including 53 species which are provided optimal habitat. In comparison, oak woodlands potentially support 153 species (optimal habitat for 26 species); chaparral is used by 141 species (42 for optimal habitat); and pine forest is used by 134 species (optimal habitat for 3 species). In summary, riparian habitat is used by more species and provides a higher percentage of optimal habitat than the other cover types.

TABLE 4.5-6 PROJECTED VEGETATIVE COVER TYPE LOSSES BASED ON VARIOUS AGGREGATE ALTERNATIVES

ALTERNATIVE	SSOW	NSOW	CHAP	PINE	RIP	BAR	GRASS	EXPOSED	TOTAL
Middle Fork									
Sand and Gravel Alternative									
1. Gravel Mining					34.0	75.0		98.0	207.0
2. Conveyor Line	16.2	3.7	7.0	1.9			3.5		32.3
3. Access Roads									
a. South Bank (LDB)	1.7	1.3	0.1	1.3					4.4
b. North Bank (RDB)	1.1	0.5	1.3						2.9
TOTAL	19.0	5.5	8.4	3.2	34.0	75.0	3.5	98.0	246.6
Old Cool Quarry									
1. Aggregate Mining									
2. Conveyor Line	6.2	3.0	3.1				3.4		15.7
TOTAL	6.2	3.0	3.1	0.0	0.0	0.0	3.4	0.0	15.7
Cool Quarry Amphibolite									
1. Aggregate Mining	3.9	15.8		11.8	7.9				39.4
2. Conveyor Line	6.2	3.0	3.1				3.4		15.7
TOTAL	10.1	18.8	3.1	11.8	7.9	0.0	3.4	0.0	55.1
Mississippi Bar									
Sand and Gravel Alternative									
1. Gravel Mining					30.0			124.0	154.0
TOTAL	0.0	0.0	0.0	0.0	30.0	0.0	0.0	124.0	154.0
Yuba Gold Fields									
1. Gravel Mining					15.5			99.0	
TOTAL	0.0	0.0	0.0	0.0	15.5	0.0	0.0	99.0	0.0
Chevreaux Quarry									
1. Aggregate Mining	245.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	245.0
TOTAL	245.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	245.0

CODES: SSOW = South Slope Oak Woodlands; NSOW = North Slope Oak Woodlands; CHAP = Chaparral; PINE = Conifer Forest; RIP = Riparian Shrub/Scrub; BAR = Gravel Bar

Therefore, with respect to relative habitat value and resource scarcity, the loss of the *Riparian Shrub/Scrub* would be substantial. Further, the California Department of Fish and Game has determined that the *Riparian Shrub/Scrub* cover type is a rare community. Also, because the predominant species comprising this community are either obligative or facultative-wet hydrophytes, such as sandbar willow, dusky willow, Fremont cottonwood, Oregon ash, white alder, etc. (Reed 1988), under CDFG guidelines (Rollins 1987), the community would constitute a wetland. Therefore, based on Significance Threshold Criteria G6 and G7, the loss of this cover type would constitute a significant impact.

Construction of the aggregate conveyor system from the dam site to Cherokee Bar would entail the clearing and grubbing of approximately 34 acres of upland vegetative communities within the right-of-way. These communities include *Oak Woodlands* (21.1 acres), *Chaparral* (7.2 acres), *Conifer Forest* (2.0 acres), and *Grasslands* (3.8 acres).

Construction of the new access road, which is an extension of the current limited access trail/road that terminates at Maine Bar, would require the removal of approximately 9.7 acres of vegetated areas, including approximately 7.2 acres of *Oak Woodlands*, 2.4 acres of *Conifer Forest*, and 0.1 acres of *Grassland* cover types.

Impacts on Wildlife. The loss of nesting, foraging, and cover habitat attendant with project implementation would result in the cumulative loss of 57.2 Habitat Units (Table 4.5-7).

Impacts on Fish and Aquatic Resources. Because excavation of the sand and gravel deposits would occur from wet pits on the gravel bars, no direct impact to fish and aquatic resources are expected. However, a consequence of flood plain gravel extraction is the creation of large pits, typically much deeper than the thalweg of the adjacent stream. The pits are isolated from the river by only a weak gravel dike. During seasonal high flows, the river can overtop or breech the levees, creating a new thalweg and isolating and/or dewatering the former channel (Dunne and Leopold, 1979; MacDonald, 1988).

An immediate impact of the dewatering of the natural channel would be the loss of the aquatic macroinvertebrates within the affected reaches. This loss would likely be short-lived since recolonization of macroinvertebrates by drift from upstream reaches is a relatively rapid process. Provided adequate substrate is available, denuded streams bottoms have been found to completely recolonize within months after disturbances (Water 1964; Hynes 1969; Griffith and Andrews 1981; and Thomas 1985). Studies on the impacts of suction dredging for gold on the North Fork of the American River found full recolonization by macroinvertebrates within 45 days after disturbance (Harvey et al. 1982; Harvey 1986).

However, as noted in Section 4.10, it is likely that any residual gravel in the stream after mining has ceased will be very much reduced and likely confined to small point bars. The remaining substrate will be a combination of exposed bedrock and boulder/cobble fields,

TABLE 4.5-7 PROJECTED WILDLIFE LOSSES, IN HABITAT UNITS, BY VEGETATIVE COVER TYPE

	Acreage Lost	HSI Value	Habitat Units Lost
Middle Fork Sand and Gravel Alternative			
1. SSOW	19.0	0.77	14.6
2. NSOW	5.5	0.59	3.2
3. CHAPARRAL	8.4	0.85	7.1
4. CONIFER	3.2	0.77	2.5
5. GRASSLAND	3.5	0.73	2.6
6. RIPARIAN	34.0	0.8	27.2
7. GRAVEL BAR	75.0	0.75	56.3
Total			113.5
Old Cool Quarry			
1. SSOW	6.2	0.77	4.8
2. NSOW	3.0	0.59	1.8
3. CHAPARRAL	3.1	0.85	2.6
4. CONIFER	0.0	0.77	0.0
5. GRASSLAND	3.4	0.73	2.5
6. RIPARIAN	0.0	0.8	0.0
7. GRAVEL BAR	0.0	0.75	0.0
Total			11.7
Cool Amphibolite Quarry			
1. SSOW	10.1	0.77	7.8
2. NSOW	18.8	0.59	11.1
3. CHAPARRAL	3.1	0.85	2.6
4. CONIFER	11.8	0.77	9.1
5. GRASSLAND	3.4	0.73	2.5
6. RIPARIAN	7.9	0.8	6.3
7. GRAVEL BAR	0.0	0.75	0.0
Total			39.4
Mississippi Bar Sand and Gravel			
1. SSOW	0.0	0.77	0.0
2. NSOW	0.0	0.59	0.0
3. CHAPARRAL	0.0	0.85	0.0
4. CONIFER	0.0	0.77	0.0
5. GRASSLAND	0.0	0.73	0.0
6. RIPARIAN	30.0	0.8	24.0
7. GRAVEL BAR	0.0	0.75	0.0
Total			24.0
Chevreaux Quarry			
1. SSOW	245.0	0.77	188.7
2. NSOW	0.0	0.59	0.0
3. CHAPARRAL	0.0	0.85	0.0
4. CONIFER	0.0	0.77	0.0
5. GRASSLAND	0.0	0.73	0.0
6. RIPARIAN	0.0	0.8	0.0
7. GRAVEL BAR	0.0	0.75	0.0
Total			188.7
Yuba Fields			
1. SSOW	0.0	0.77	0.0
2. NSOW	0.0	0.59	0.0
3. CHAPARRAL	0.0	0.85	0.0
4. CONIFER	0.0	0.77	0.0
5. GRASSLAND	0.0	0.73	0.0
6. RIPARIAN	15.5	0.8	12.4
7. GRAVEL BAR	0.0	0.75	0.0
Total			12.4

similar in appearance to the Ruck-A-Chucky reach. As such, interstitial areas for macroinvertebrate colonization would be greatly reduced.

Trout, which are the principal game fish within the affected reach, generally require 18-24 inches of appropriately sized gravels, free of fines, to facilitate intergravel flow and exchange of oxygen for eggs within the redd. Loss of gravels would reduce any potential spawning within the affected reach. Spawning has not been observed within the affected reach; however, the area may be used for feeding. The reduction of macroinvertebrates, which are the principal prey base of rainbow trout, brown trout, and brook trout, would adversely impact these species. Assuming a minimum density of 100 trout per mile, the project could result in the reduction of the trout population by 600 individual fish. Similar decreases in other gravel-dependent species, such as riffle sculpin, Sacramento sucker, and hardhead would occur with a corresponding increase in rocky bottom inhabitants, such as the Sacramento squawfish and California roach.

Because the project would substantially diminish habitat for fish, wildlife, or plants (Significance Threshold D), particularly resident trout populations that utilize the 6 mile reach for spawning, rearing, and feeding habitat, the loss would constitute a significant unavoidable adverse environmental impact (Class I).

Impacts on Sensitive Species

Bald Eagle. Bald eagles have not nested in the American River watershed area in recent years; however, sightings of wintering bald eagles around Folsom Reservoir are fairly common. Occasionally, eagles are observed along the lower American River. Bald eagles are observed less frequently above Folsom Reservoir. Ground and aerial surveys conducted in winter/spring 1989-90 did not observe any wintering eagle between the lower American River and the upper watershed (approximate elevation 1,200 feet, msl). Construction activities and increased human presence could result in the disturbance and temporary displacement of roosting and/or foraging individuals; however, no long-term significant adverse impacts on nesting and/or wintering habitat of the bald eagle would result from project implementation.

American Peregrine Falcon. The peregrine falcon is known to nest and overwinter in the western Sierra Nevada. Typical nesting habitat includes nest on a ledge of large cliff faces; however, the use of tall trees and buildings has been recorded. Nesting and wintering habitats are variable and include wetlands, woodlands, other forested habitat, cities, agricultural areas, and coastal habitats (CDFG 1990). The project is unlikely to impact nesting habitat; however, removal of vegetation, particularly *Riparian Shrub-Scrub*, may affect the prey base by reducing available habitat of birds preyed upon by the peregrine falcon. This impact is considered adverse but not significant (Class III).

Valley Elderberry Longhorn Beetle. Clumps of elderberry plants were found along the gravel bars along the Middle Fork. These plants are the host plant for the valley elderberry

longhorn beetle (VELB), a federally listed threatened species. Examination of the plants for signs of the VELB (adults, frass, emergence holes) was conducted by the U.S. Fish and Wildlife Service (1991) and during field reconnaissance surveys in July of 1991. In each survey, signs of VELB inhabitation were not found; however, presence of suitable habitat offered by the plants should presume existence of the beetle (U.S. Fish and Wildlife Service 1991). In accordance with Significant Threshold B, removal of elderberry plants, secondary to gravel extraction, would constitute a mitigable significant adverse environmental impact (Class II).

Tricolored Blackbird. There are no reports of nesting colonies in Placer or El Dorado Counties. Suitable breeding habitat for the tricolored blackbird includes emergent wetland vegetation (dense cattails or tules) or willow, wild rose, and blackberry thickets near fresh water. Feeding habitat includes agricultural fields and grasslands. The project site offers limited habitat value based on the absence of preferred foraging habitat (U.S. Fish and Wildlife Service 1991). Consequently, the project is unlikely to have any significant impact on the species.

California Spotted Owl. The California subspecies (*Strix occidentalis occidentalis*) occurs in the Sierra Nevada and mountainous areas of Southern California. It is currently listed as a Category 2 candidate, in contrast to the northern subspecies (*S. o. caurina*), which is a federally listed endangered species (U.S. Fish and Wildlife Service 1991). Studies have documented the downslope migration to winter ranges as low as 885 feet in the Placerville/Auburn area (U.S. Fish and Wildlife Service 1991). All impacts associated with the gravel bar mining occur below the 800-foot elevation and would not likely result in the disturbance to winter range.

California Red-legged Frog. The red-legged frog requires quiet, permanent pools of streams and marshes with extensive vegetation for escape cover. Potential habitat in the project area include backwater areas and isolated permanent and seasonal ponds with emergent vegetation, isolated ponds, and canals and drainages that lack bullfrogs or other large aquatic predators (U.S. Fish and Wildlife Service 1991). Elimination of backwater areas within the gravel bars could potentially reduce habitat of the red-legged frog.

Gabbroic Northern Mixed Chaparral Endemics. The project is unlikely to impact any of the species associated with the Gabbroic Northern Mixed Chaparral community or serpentine soils since neither exist in areas which may be affected by the proposed project, nor have the species been observed during vegetative reconnaissance surveys.

Bisbee Peak Rush-rose. The Bisbee Peak rush-rose is an inhabitant of open, dry chaparral and oak-pine woodland, on olivine schist, gabbro, or serpentine soils. It has not been recorded in the project area, nor was the species observed during reconnaissance surveys. The northernmost report of the species is within the Pilot Hill area (Smith and Berg 1988). Therefore, the project is not likely to impact the species.

Stebbin's Phacelia. Stebbins' phacelia has not been reported in the project area, nor found during vegetative surveys. It has been recorded in El Dorado and Placer Counties, but at higher elevations (3000-4800 feet) within the lower montane coniferous forest (Smith and Berg 1988). Based on this, the species is unlikely to be impacted by the project.

Spiny Rhyacophilan Caddisfly. The spiny rhyacophilan caddisfly has been reported from Placer County in Ladys Canyon of the Middle Fork American River watershed. The species occupies cool, running water and is presumed extant in the vicinity of the project. The project could potentially impact this species. Therefore, consistent with Significance Threshold F1, project implementation could result in a Class I impact on the caddisfly.

Old Cool Quarry (Spreckles)

Impacts on Vegetation. No loss of vegetation would occur as a result of quarry operations; however, construction of access roads and conveyor lines would result in the loss of 9.2 acres oak woodlands, 3.1 acres of chaparral, and 3.4 acres of grasslands (Table 4.5-5). This loss of habitat would constitute a Class III impact.

Impacts on Wildlife. Cumulative wildlife losses associated with the loss of the cover types described in Table 4.5-5 would total approximately 11.7 Habitat Units, and constitute a Class III impact.

Impacts on Fish and Aquatic Resources. No long-term significant impacts to fish and aquatic resources would result from implementation of the alternative. Short-term impacts resulting from construction of the conveyor line would likely create temporary turbid conditions which would impair aquatic organisms.

Impacts on Sensitive Species. The Yate's snail is an inhabitant of limestone cave/outcrops and has been report within the same section as the Old Cool Quarry. However, the on-going disturbance at the site resulting from aggregate mining has minimized the likelihood of the species occupying the quarry.

Cool Quarry Amphibolite

Impacts on Vegetation. Development of the new quarry at the Cool Amphibolite site would result in the loss of approximately 55.1 acres of vegetation. Approximately 48 acres would be lost due to aggregate mining, including 19.7 acres of *Oak Woodlands*, 11.8 acres of *Conifer Forest*, and 7.9 acres of *Riparian Shrub/Scrub*. An additional 15.7 acres of vegetation would be lost as a result of the construction of the conveyor line, including 9.2 acres of *Oak Woodlands*, 3.1 acres of *Chaparral*, and 3.4 acres of *Grassland* (Table 4.5-6).

In accordance with Significance Criteria G6, loss of *Riparian Shrub/Scrub* cover would constitute a Class II impact.

Impacts on Wildlife. The loss of 55.1 acres of various vegetative communities would result in the aggregate loss of 39.4 Habitat Units (Table 4.5-7).

No significant loss of wildlife would result from this alternative, consequently, implementation would result in a Class III impact.

Impacts on Fish and Aquatic Resources. No fish or aquatic resources would be impacted by this project, consequently no significant impacts are anticipated.

Impacts on Sensitive Species. No sensitive species have been identified in the proposed quarry site, consequently no significant impacts are anticipated.

Bear River and Chevreaux Quarry

Impacts on Vegetation. Due to the lack of interest from the Chevreaux quarry to supply aggregate for the project, specific information concerning aggregate extraction was not available. However, in order to approximate impacts, for the purpose of this analysis, it was assumed that new areas would be quarried to obtain the aggregate. As a result, approximately 245 acres of oak woodlands would be removed to secure sufficient quantities to construct the dam (Table 4.5-6). Loss of over 200 acres of *Oak Woodlands* would substantially diminish habitat for plants (Significance Criteria D), and would constitute a Class II impact.

If sufficient materials would be available from the combination of wet pit dredging in Lake Combie/Bear River and hardrock quarry mining from the present quarry site, impacts to vegetation would be much reduced since both of these areas have been cleared of vegetation.

Impacts on Wildlife. The loss of 245 acres of *Oak Woodlands* would result in the loss of approximately 189 Habitat Units (Table 4.6-7), and constitute a Class II impact (Significance Criteria D).

Wildlife impacts could be reduced to a less than significant level (Class III) if existing hardrock quarry or flood plain excavation sites were used and vegetated sites were minimized.

Impacts on Fish and Aquatic Resources. Mining of aggregate from the hardrock quarry would not result in any impacts to fish or other aquatic resources.

Impacts on Sensitive Species. No sensitive species have been identified in the vicinity of the Joe Chevreaux Quarry.

Mississippi Bar Sand and Gravel Deposits

Impacts on Vegetation. Project implementation could result in the loss of 30 acres of riparian woodland (Table 4.5-6). Based on Significance Criteria G6, this would constitute a significant adverse impact that can be feasibly mitigated (Class II).

Impacts on Wildlife. Utilization of the Mississippi Bar Sand and Gravel Deposits as an aggregate source would result in the loss of 24 Habitat Units associated with the loss of 30 acres of riparian shrub/scrub habitat (Table 4.5-7).

A great blue heron rookery, located in a stand of digger pines, has been recorded on Mississippi Bar (California Department of Fish and Game 1991). Human disturbance has been recognized as a factor affecting the successful nesting and fledging of great blue herons (Murphy 1988; Quinney 1983; and Werschkul et al. 1976). However, the nature of the human disturbance is critical in the response elicited by the birds. For example, a walk through a rookery can result in temporary nest abandonment which increases the vulnerability of eggs and nestlings to predation, while logging and other habitat destruction can result in total rookery abandonment. In general, disturbance outside a rookery results in a contraction of nesting sites away from the disturbance.

Werschkul et al. (1976) observed that the average distances from a point of disturbance to inactive and active nests were approximately 485 feet and 710 feet, respectively. This suggests that a threshold response distance is maintained. Other studies have reported successful colonies in areas of relatively high human activity (Murphy 1988). These included colonies within 100 meters of a public boat launching ramp and in an area of high powerboat use and waterskiing; a colony in an area surrounded on three sides by development; and a small rookery located within 50 meters of a condominium development. Also, a successful heronry exists in a eucalyptus grove within the parking lot of a busy restaurant and state park beach in Goleta, California. Despite high and persistent levels of noise, lights, traffic, and human activity directly beneath nesting sites, the herons continue to use the site.

Significant impacts to the heronry could occur if minimum buffer distances between aggregate mining operations and the rookery is not maintained during the breeding season, or it nest trees are removed during mining.

Impacts on Fish and Aquatic Resources. No significant impacts of fish and/or aquatic resources are expected under this alternative.

Impacts on Sensitive Species. The VELB has been found along the lower American River from the Lake Natoma to Goethe Park. Evidence of VELB inhabitation has been found in elderberry plants on the Mississippi Bar. The project could potentially impact VELB populations. Cooper's hawk, a California Species of Special Concern and Fully Protected Species is known to nest on Mississippi Bar and could potentially be impacted by the project. Therefore, based on Significance Criteria B and F5, implementation of this alternative would constitute significant (Class II) impacts.

Yuba River Dredge Fields

Impacts on Vegetation. As noted in Section 4.5.1, vegetation within the Yuba River Dredge Fields is sparse and the precise location of extraction is not known at this time. However, assuming an average depth of 25 feet, approximately 155 acres would be disturbed to acquire sand and gravel. Assuming further that based on analysis of aerial photographs and on-site visual assessment that, on the average, vegetation covers approximately 10 percent of the site, a total of 15.5 acres of riparian habitat would be lost (Table 4.5-6). In accordance with Significance Threshold G6, the loss of riparian habitat would constitute Class II impact.

Impacts on Wildlife. Approximately 12.4 Habitat Units would be lost as a result of the loss of 15.5 acres of riparian habitat. This would constitute a Class III impact.

Impacts on Fish and Aquatic Resources. No impacts on fish and/or aquatic resources would be likely under this alternative.

Impacts on Sensitive Species. Colonies of tricolored blackbirds have been reported nesting in the Yuba Gold Field dredger pits in 1934, and are presumed extant (California Department of Fish and Game 1991). Loss of potential breeding habitat would be considered a Class II impact.

4.5.3 MITIGATION MEASURES

The primary purpose of this report is to identify the least environmentally damaging practicable alternative. The identification of this alternative has not been assessed on the basis of mitigable impacts, rather a straight-up comparison of impacts. The utilization of mitigation measures could potentially lower impacts of a certain alternative to less than significant impacts, or even convert a less-preferable alternative to the environmentally superior alternative.

The principal goal of mitigation is avoidance of impacts when feasible, followed by the minimization of impacts. If such measures are infeasible, mitigation through replacement or rectification is considered. The preferred method of replacement of lost environmental

values, particularly biological resources, is to replace the lost values with in-kind resources at or very near the impact site. Less preferable is out-of-kind replacement (e.g., oak woodlands for riparian forest) and off-site mitigation.

Mitigation measures identified in this section are conceptual and are described only in terms of feasibility, considering availability of lands, technical application, past success, and economics.

Middle Fork Sand and Gravel Deposits

Vegetation. The loss of *Riparian Shrub/Scrub* results from the loss of the gravel bars and suitable substrate for establishment. With the low predicted gravel recruitment, substantial recover could not be expected for over 50 years. Therefore, on-site replacement is not feasible. Off-site mitigation (e.g., reaches within the watershed, or within other river systems) could provide an opportunity to mitigate for vegetative losses associated with this aggregate alternative by renovating existing degraded stream segments to recover the lost acreage of riparian habitat. Sufficient stream renovation to replace approximately 34 acres of *Riparian Shrub/Scrub* and 75 acres of *Gravel Bar Scrub*.

The temporary loss of upland cover types resulting from new road construction and conveyor construction could best be mitigated by rehabilitating the sites (e.g., regrading, replanting, etc.) upon completion of the project.

Wildlife. Wildlife losses associated with the removal of Riparian Shrub/Scrub and Gravel Bar Scrub could not be feasibly mitigated at the project site due to the lack of adequate substrate to replant. As such, off-site mitigation would be the feasible mitigation strategy.

Fish and Aquatic Resources. Extraction of 75 percent of the existing sand and gravel in the flood plain would result in long-term losses/displacement of gravel-dependent fish and aquatic resources within the affected reach. There is no method of rehabilitating the impacted reach short of replacing lost gravel, which would be infeasible.

Potential in-kind mitigation could include developing a put-and-take fishery within the affected reach by planting catchable-size, hatchery-reared rainbow and brown trout. Survival of the planted trout would be dependent on the amount of invertebrate drift to the reach; however, reproduction of the introduced stock would be very unlikely due to the lack of sufficient spawning gravels and water temperatures in the affected reach.

Potential off-site mitigation would include renovation/enhancement of degraded stream segments within the Middle Fork, or in other nearby watersheds. Such measures could include improvement of salmonid habitat through the construction of overhangs for cover; erosion- and grade-control structures, creation of pools or runs; artificial spawning channels; improvement of passage, etc. (Nelson et al. 1978; Ministry of Environment 1980; Bell 1986).

Sensitive Species. The only listed species that could potentially inhabit area to be impacted by the project is the Valley Elderberry Longhorn Beetle, a federal Threatened species. Although the VELB has not been recorded in the area, the presence of elderberry plants presumes existence of the species (U.S. Fish and Wildlife Service 1991).

Because neither the VELB or the host elderberry plant are stream obligates, on-site mitigation is a feasible measure. Existing elderberry plants could be transplanted in accordance with U.S. Fish and Wildlife Service requirements, or new plants could be propagated in sufficient numbers to replace lost stand acreage.

Old Cool Quarry (Spreckles)

Vegetation

Because of the lack of vegetation within the quarry area, no mitigation would be required. However, the temporary loss of approximately 16 acres of upland cover types resulting from new road construction and conveyor construction could best be mitigated by rehabilitating the sites (e.g., regrading, replanting, etc.) upon completion of the project.

Wildlife. Rehabilitation of roads and conveyor routes would restore lost wildlife habitat values.

Fish and Aquatic Resources. No mitigation required.

Sensitive Species. No mitigation required.

Cool Quarry Amphibolite

Vegetation. Loss of 7.9 acres of Riparian Shrub/Scrub and 39.4 acres of upland cover types would occur as a result of clearing for aggregate mining. An additional loss of 15.7 acres of upland cover for the construction of roads and conveyor routes would occur, as described above.

On-site mitigation, as a component of a reclamation plan would be of limited efficacy in reestablishing lost acreage. This is due to the fact that significant volumes of the slope would be removed to secure the aggregate. Even if all topsoil was stockpiled and the site renovated, the remaining acreage would be less than the pre-excavation conditions.

If, however, the material removed from the existing slide below the dam was stockpiled and used to fill the amphibolite quarry site, additional acreage for quarry rehabilitation would be available.

Off-site mitigation, such as rehabilitation of disturbed areas (e.g., former pasture) could be used to recover lost cover type acreage.

Wildlife. The loss of wildlife values, through the combined loss of 39.4 Habitat Units from the quarry operation and road and conveyor route impacts, would be recovered by implementation of an appropriate revegetation plan utilizing on-project and/or off-project sites.

Fish and Aquatic Resources. No mitigation required.

Sensitive Species. No mitigation required.

Bear River and Chevreaux Quarry

Vegetation. The Chevreaux Quarry has existing authority, including environmental documentation prepared pursuant to CEQA, to quarry on approximately 478 acres in Placer County and 680 acres in Nevada County. The estimated 245 acres of Oak Woodland cover that would be removed to obtain the aggregate necessary to construct the dam, could be contained within the approved quarry site. As a result, if the Chevreaux Quarry aggregates were used, mitigation would fall within the purview of the approved reclamation plans prepared pursuant to SMARA (Joe Chevreaux Company 1986a; 1986b).

Initial plans are to convert the quarry site into a landfill, if feasible, because the site is adjacent to the present Placer County landfill. Upon closure of the landfill, the site would be revegetated. If the landfill proposal is unacceptable, other potential uses of the site, as identified in the reclamation plan, include Christmas tree farming, indoor and outdoor storage, farming, grazing, raising of poultry, or timber production.

Wildlife. Mitigation for the loss of wildlife secondary to the loss of Oak Woodlands would be within the approved reclamation plan for the quarry.

Fish and Aquatic Resources. No mitigation would be required.

Sensitive Species. No mitigation would be required.

Mississippi Bar Sand and Gravel Deposits

Vegetation. The U.S. Bureau of Reclamation (1988) prepared and approved an Environmental Commitment Plan to rehabilitate Mississippi Bar.

A principle goal of the plan is to "...restore the site to the biological productivity present before dredger tailings were dumped on the site." The Bureau of Reclamation has

developed a plan to grade and revegetate the bar with native shrubs and trees upon cessation of aggregate mining activities.

Execution of the plan will compensate for losses incurred as a result of aggregate excavation.

Wildlife. Maintenance of adequate distance (1000 feet) from the heronry during breeding and rearing periods would minimize impacts to great blue herons nests on Mississippi Bar.

Rehabilitation and revegetation of the disturbed areas will provide the requisite wildlife habitat lost as a result of gravel extraction activities.

Fish and Aquatic Resources. No specific mitigation for fish and aquatic species would be required.

Sensitive Species. Avoidance of all elderberry plants during gravel extraction activities would prevent the loss of VELB habitat.

Maintenance of adequate distance between gravel extraction operations and nesting sites during nesting and rearing periods would minimize impacts to Cooper's hawks.

Yuba River Dredge Fields

Vegetation. Implementation of the revegetation components of Reclamation Plans for specific gravel extraction areas would compensate for the potential losses of vegetation resulting from adoption of this alternative.

Wildlife. Mitigation for the loss of wildlife secondary to the loss of riparian habitat would be within the approved reclamation plan for the quarry.

Fish and Aquatic Resources. No specific mitigation for fish and aquatic resources is necessary.

Sensitive Species. Avoidance of marsh and other potential nesting sites would minimize impacts to the tricolored blackbird.

4.6 TRANSPORTATION

This section provides an overview of existing transportation systems which may serve to transport materials from the various alternative aggregate sources to the dam site. Of central interest are aggregate transport-related impacts on the local and regional transportation network. Aggregate transport by truck can result in impacts to roadway capacity, roadbed structure, and traffic safety.

4.6.1 EXISTING CONDITIONS

All six of the potential aggregate sources are located in the central foothills of the Sierra Nevada (Figure 4.6-1). Topography ranges from gently rolling (in the case of the distant sources) to steep and precipitous (Middle Fork bars and the quarry sites). Roadways accessing the various alternative sources range from two-lane rural residential roads and streets to 6-lane interstates. The following sections provide cursory descriptions of possible transportation modes and routes to the dam site.

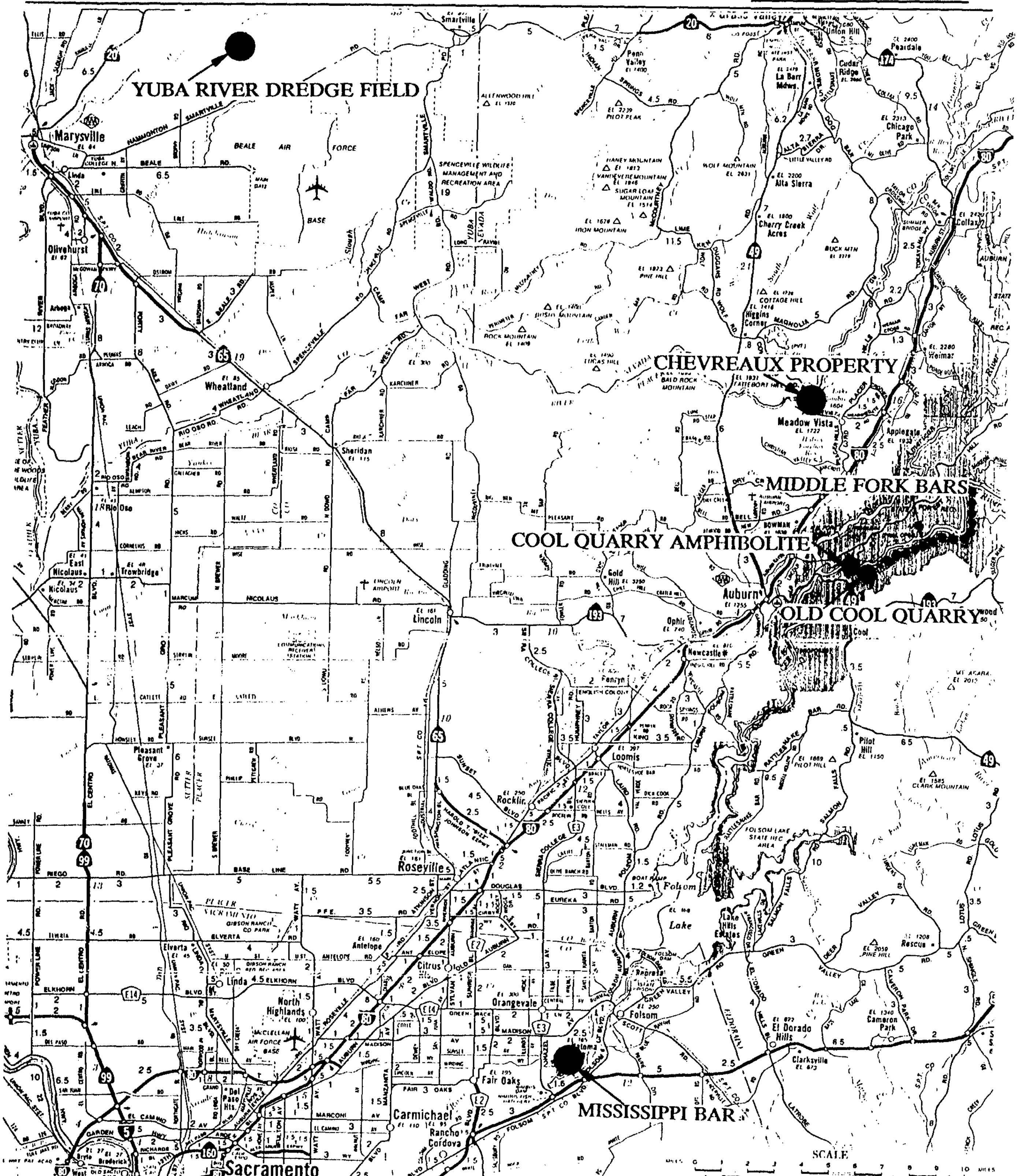
Middle Fork Sand and Gravel Deposits

Figure 3-21 shows hypothetical access roads and conveyor routes necessary to implement the Middle Fork bar alternative. Temporary roads would be required to provide access to the excavation and processing sites. These routes would connect to existing county or state roads (Figure 4.6-2) and would allow work crews with their accompanying equipment to access the 7-mile stretch of canyon bottom. Reconstruction of existing unimproved routes in the canyon would include widening to a minimum of 25 feet with grading and filling as necessary. A 15-20 foot right of way would be necessary to accommodate the primary conveyor which would ultimately run the 7-mile length of the canyon bottom (Figure 3-21). Improvements and construction of these routes would occur as excavation progressed upstream. Access to the processing plant would need to be wide enough to allow travel of employee vehicles and large pieces of construction and mining equipment. Roads to the processing plant would need to be developed prior to plant construction.

Old Cool Quarry (Spreckles)

As with the Middle Fork bar alternative, access and transport of material from Old Cool Quarry would involve construction of access roads and conveyor routes to an aggregate processing facility and the dam site itself (Figure 3-21). Although truck transport may not be the most efficient method of conveyance, existing dirt and gravel roads within the Auburn SRA could be improved or new roads constructed to transport material down to river level for stockpiling or immediate conveyance to the dam site vicinity. Alternatively, temporary conveyors could be configured in such a way so as to minimize truck transport of the material.

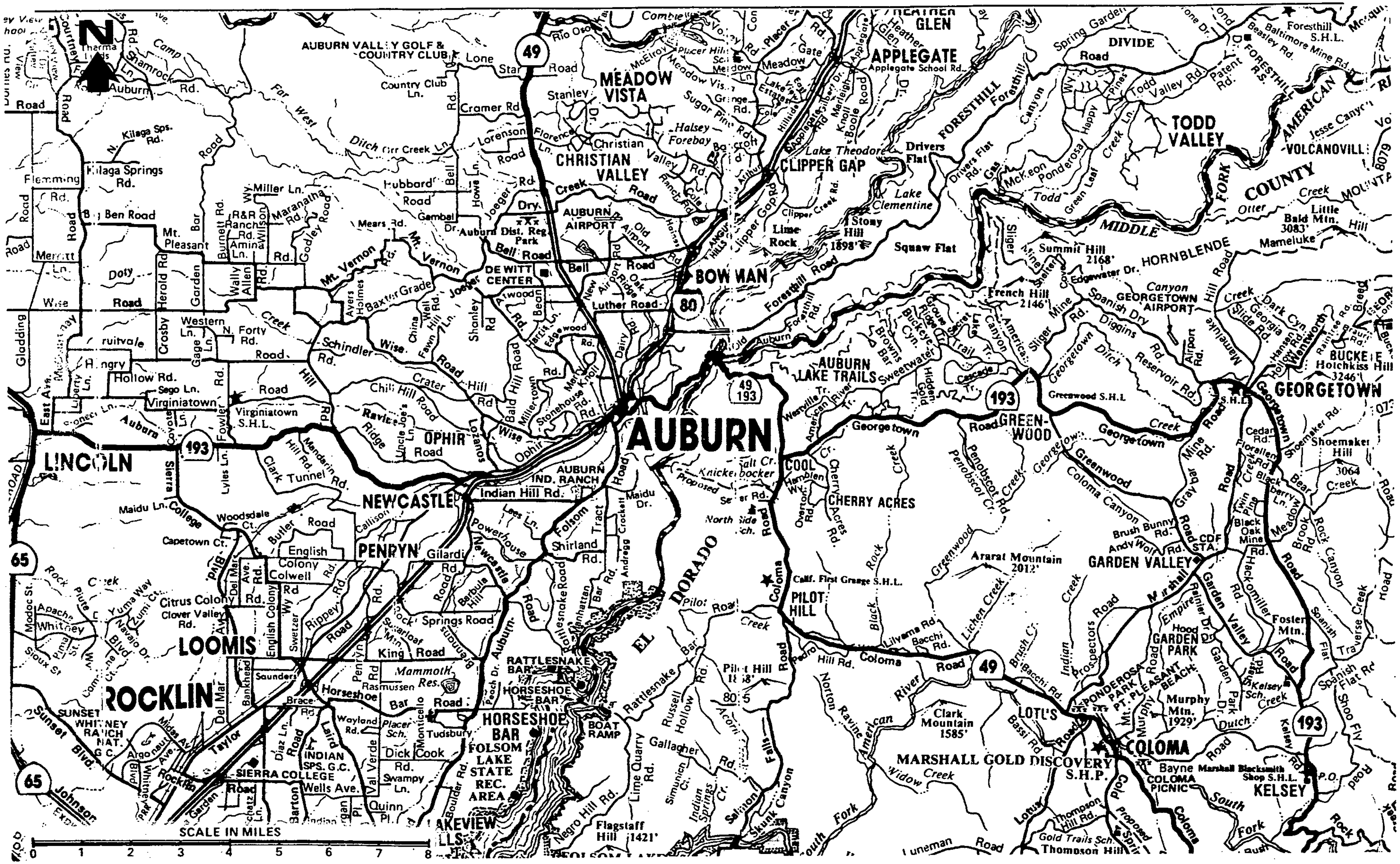
FUGRO McClelland



Source Location Map

FIGURE 4.6-1

FUGRO McClelland



Local Roadway Network

FIGURE 4.6-2

Depending on positioning of mining and processing facilities, distance from the Old Cool Quarry downstream to the dam site is about 5 miles. The portion of the quarry currently operating is approximately 800 vertical feet above the Middle Fork. Movement of processed material to the dam site could involve trucks, conveyors or a combination of both. Material could be quarried and processed near the elevation of the current working face (1400 feet) then trucked or conveyed to interim storage areas at river elevation. Alternatively, material could be quarried, processed and stored at river elevation for later conveyance to its point of use.

A key consideration is prospective haul routes. Movement of material downhill to river elevation via trucks would involve considerable improvement and expansion of the old quarry access routes and existing fire roads. Because of the large volume of traffic, a possible configuration may be a singe-lane loop route designed to accommodate substantial numbers of 50 to 80-ton pit trucks. Depending on the circumstances, this type of vehicle could accommodate maximum grades of 4-8 percent.

Rail transport may also be an option for this source. Beginning in 1912, limestone was brought up from the canyon to the SP line at Flint, about a mile below Auburn on the westbound track, over a winding 8-mile standard gauge railroad (Signor, 1985). Known as the Mountain Quarries Railroad, it required a switchback, 18 trestles, and a concrete viaduct over the river. The concrete viaduct is still in place. To move material up out of the canyon to Flint, the locomotive could only handle three loads at a time.

Cool Quarry Amphibolite

The circumstances surrounding the transport of aggregate material from the Cool Quarry Amphibolite site are similar to those of the Old Cool Quarry. Horizontal and vertical distances to the dam site are roughly equal (Figure 3-22). Creation of new roads as well as expansion and improvement of existing roads would be necessary.

Bear River and Chevreaux Quarry

Access to the Chevreaux property is provided from I-80 via Placer Hills and Combie Roads (Figure 4.6-1). Placer Hills Road is a narrow 2-lane winding rural road that is the primary link to I-80 for the town of Meadow Vista. The haul route would pass through the central business district with existing school and commercial developments. Similarly, haul routes would proceed through Auburn via surface streets with varying design capacities and structural characteristics. One-way, the haul distance from the Chevreaux property to the dam site is about 11 miles. Rail transport is not feasible (Corps, 1991).

Movement of 6.75 million cubic yards to the dam site by 25-ton highway transport trucks via Placer Hills Road and I-80 would involve significant numbers of trucks. According to the Corps' Special Aggregate Report, 700 to 900 truck trips would be needed to deliver

aggregate each day or about 40 truck loads per hour. The current average daily traffic count along Placer Hills Road is 8,600 (Dondro, pers. comm., 1991).

Mississippi Bar

Most material transport to and from the current aggregate operation at Mississippi Bar (Teichert's) is via Main Avenue (Figure 4.6-3). The majority of trucks leaving Teichert travel northeast on Main Avenue to Madison Avenue, a major east-west arterial which links Auburn-Folsom Road and I-80 (Figure 4.6-1). Depending on destination, a portion of the trucks utilize a steeper route to U.S. 50 by traveling west on Sunset Avenue to Hazel.

Distance between the dam site and Mississippi Bar is about 18 miles via Auburn-Folsom Road. Historic Auburn-Folsom Road is a steep, winding 2-lane rural highway. Both Sunset and Main Avenues are collector streets which travel through residential neighborhoods. Hazel Avenue is a major 4-lane arterial which runs north through Orangevale and south to U.S. 50. Madison Avenue is a 4 to 6-lane arterial which is a direct route to I-80, a distance of about 6 miles.

Once existing stockpiles are depleted, truck traffic to and from Teichert's aggregate plant is limited by plant production. The current maximum production rate is 300 tons per hour. Using 25-ton highway trucks, this represents 12 deliveries per hour. A delivery rate of 40 trucks per hour (Corps, 1991) would mean an increase in truck traffic of over 300 percent.

Rail transport of material from Mississippi Bar has been identified as a possible method of delivery. To utilize existing lines, material would need to be railed down the existing Southern Pacific (SP) track which parallels Folsom Blvd into Sacramento. Minimal switching would be necessary to route the train east via the SP line which parallels I-80 to SP's hub in Roseville. Roseville is the divisional hub for all of northern California and trains moving through the hub must change crews. From Roseville the aggregate could be railed up the SP line to the Auburn vicinity.

Yuba River Dredge Fields

One-way distance from the Yuba River dredge fields to the dam site is approximately 40 miles. A potential truck route from the Western Aggregate operation to the dam site would be via Hammonton-Smartville Road through Linda to Highway 65 south via North Beale Road. This route would circumvent Beale Air Force Base (Beale AFB). At the town of Lincoln, the route would connect to State Route 193 (SR 193) which connects to I-80 just north of Indian Hill Road. These routes are primarily 2-lane rural roads. For this scenario, trucks would transit congested commercial districts in Linda, Wheatland, Lincoln and Auburn. Much of the route has adjacent residential uses.

Rail transport is a viable option for this alternative. Western Aggregate, the largest aggregate producer in the area, is in the process of negotiating a direct route south through

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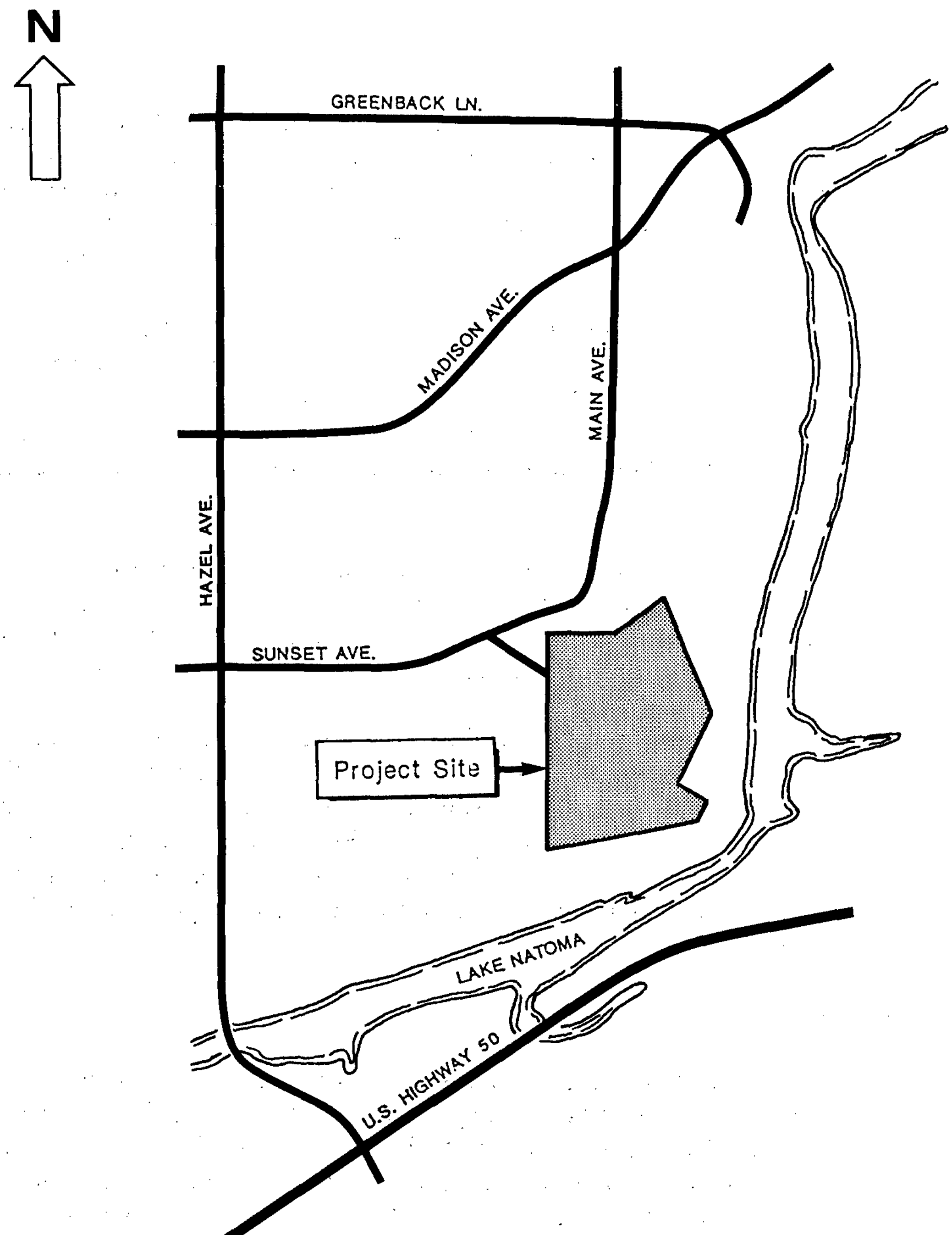


Mississippi Bar - Local Roadway Configuration

FIGURE 4.6-3

Beale AFB. This spur would provide a direct link with the SP rail line which parallels SR 65. The train would route through the SP divisional hub at Roseville eventually switching to the line which runs to Auburn.

4.6.2 IMPACT ANALYSIS

The relative significance of transportation impacts for the six alternative aggregate sources depends on the location of the source, the mode of transport and the route between the source and the dam site. As of this writing, no specifics regarding transport modes and routes had been ascertained. However, some general conclusions can be drawn regarding the relative magnitude of impacts postulated for the various sources.

Distant Sources

Because of the large numbers of highway trucks involved with transport of material from a distant site, the primary consideration will be impacts to affected transportation facilities. All three of the distant sources, Chevreaux, Mississippi Bar and the Yuba River would involve truck transport over the public roadway network. Sensitive adjacent land uses aside, roadbed-related impacts would be the greatest for the most distance source, Yuba River. Next would be Mississippi Bar, followed by Chevreaux. The ability of any existing facility to withstand the necessary use (800 deliveries per day) is questionable. Without exception, truck transport of aggregate on public roadways would reduce the useful life of all affected roadbeds. This constitutes a significant unavoidable impact (Class I).

Also at issue is roadway capacity. During the construction phase, affected facilities could expect significant level of service deterioration. The duration of the temporarily decreased level of service could be as long as three years. The largest roadways, such as I-80, would be affected least. Commercial districts in areas such as Meadow Vista, Linda, Lincoln, and Auburn would be especially affected during peak traffic hours. The additional traffic could also present a threat to public safety in these congested areas. This constitutes a significant unavoidable impact (Class I).

Utilization of rail transport would eliminate road-related transportation impacts. Besides economy of scale (a typical rail car holds 100 tons), rail cars can act as temporary storage facilities, a necessary consideration due to the lack of stockpile space within the canyon. Impacts to roadbed structure and capacity would no longer be an issue. Because rail alignments are generally located in remote areas away from congested commercial and residential districts, related transportation impacts such as noise and safety would be lessened. Also, rail transport would likely have less effect on air quality (see Section 4.4, Air Quality).

However, rail transport is not without difficulties. Aggregate material has a minimal value per weight ratio and short hauls requiring multiple switching are not cost-effective. These factors affect the economic viability of rail transport. Also affecting the economics is the

cost of inter-divisional transfer through the Roseville hub which requires a labor union-mandated crew change. Because of the heavily urbanized surroundings, railing large quantities of material from Mississippi Bar via the SP line along Folsom Boulevard would have severe environmental consequences which would constitute significant unavoidable impacts (Class I).

Because large quantities of heavy material must be moved, the tracks utilized must be prepared to industrial standards or better. A one-percent grade is an approximate maximum for a heavily loaded train. It is likely the track along Folsom Boulevard would need to be rebuilt to withstand the added use. The SP line parallelling SR 65 is built to industrial standards so minimal modification would be necessary. Issues regarding ownership of equipment such as cars and locomotives as well as scheduling of deliveries would need to be resolved well in advance of project implementation.

Nearby Sources

Three of the potential sources are within close proximity to the dam site: the Middle Fork bars, Old Cool Quarry and Cool Quarry Amphibolite. The nearness of these sources allows modes of transport other than highway trucks and trains to be considered (e.g. conveyors and pit trucks). This reduces the severity and extent of potential impacts related to usage of the local and regional transportation network. Large-scale mining along the canyon bottom would create additional daily trips on SR 49 and SR 193. Regardless of the transport mode, some provision will need to be made for crossing of SR 49. Transport of aggregate from Old Cool Quarry to the dam site will require crossing the highway at some point. SR 49 will need to be rerouted during the operation or some type of over/underpass will need to be constructed. The over/underpass could either serve trucks or a conveyor system.

Some amount of impact on the public roads would result from construction traffic during the mining operation. Equipment, conveyors, generators, tanks and fuel would need to be hauled in and out of the canyon on a regular basis. For the Middle Fork sand and gravel option, seasonal gearing down of operations during the winter would create a temporary decrease in the level of service on local roads, particularly SR 49.

Ideally, because of the associated impact potential, particularly in the areas of air and water quality, minimization of truck usage is crucial. The bulk of material movement for the Middle Fork bars and the two quarry options could be accomplished by conveyors. These temporarily erected conveyors could transport material from the excavation site to the processing plant and from there to the dam site. Upon completion of the construction phase, they could be dismantled and removed. The primary impact to public transportation facilities would be decreased levels of service on I-80, SR 49 and Auburn city streets during installation and dismantling of the processing and conveyance facilities. Considerable amounts of heavy equipment activity would be necessary during these phases. The steep grades down into and out of the canyon will exacerbate traffic delays on SR 49 unless a

detour is enabled. These impacts, while adverse, are not considered significant due to their short-term, intermittent nature (Class III).

4.6.3 MITIGATION MEASURES

The primary mitigation for aggregate-related transportation impacts is choice of source. Trucking material from any of distant sites would create significant impacts on local and regional facilities. Of the three distant sources, utilization of the Yuba River source with its rail potential would minimize roadway impacts the most. If material could be railed to a suitable location near the dam site, conveyors could be used to transport material down into the canyon.

Usage of one of the nearby sources would also serve to minimize impacts to the public roadway network to levels of insignificance. Transport of material would be off-highway via conveyors or roads constructed for the project. Some amount of impact would result from start-up and shutdown phases but not nearly as much as if the bulk of the material necessary for dam construction is hauled over public roads.

Additional impact reductions can be achieved by using an aggregate transport route other than that proposed for the Middle Fork sand and gravel deposits. Material could possibly be transported to the dam site by truck or conveyor from the Old Cool Quarry or the Cool Quarry Amphibolite via an overland route rather than along the environmentally sensitive Middle Fork of the American River. If such a routing proved feasible, significant reductions in potential impacts to water, biological, recreational, and visual resources would be possible. The practicality of such a route would need to be determined during the Preliminary Engineering and Design (PED) phase of the project.

4.7 NOISE

The purpose of this section is to identify existing and projected noise levels for the various aggregate source alternatives, and to recommend noise mitigation measures for significant noise impacts. The information contained in this section was derived from published reports and observation of the noise environment surrounding the various sources.

4.7.1 EXISTING CONDITIONS

Regulatory Guidelines

Noise impacts were assessed at each of the aggregate sources by comparing project-generated construction and operational noise levels, to no-project noise levels and to the criteria and standards contained in the applicable planning documents. The noise standards which apply to the project are listed below:

Sacramento County	-	50-70 dBA - day, and 45-65 DBA - night.
Placer and El Dorado County	-	60 dB L_{dn}.
Yuba County	-	60 dB L_{dn}.

For purposes of this section, noise impacts are considered significant if project-generated noise levels would exceed the above adopted noise standards in areas of sensitive receptors.

Noise is often described as unwanted sound, and thus is a subjective reaction to characteristics of a physical phenomenon. Researchers have generally agreed that A-weighted sound pressure levels (sound levels) are well correlated with subjective reaction to noise. Variations in sound levels over time are represented by statistical descriptors, and by time-weighted composite noise metrics such as Day-Night Average Level (L_{dn}). The unit of sound measurement is the decibel (dB), sometimes expressed as dBA. Throughout this discussion, A-weighted sound pressure levels will be used to describe environmental noise unless otherwise indicated.

Community Reactions to Noise

The most frequent complaint the public makes regarding nearby mining operations is about blasting noise. Some quarries almost continually receive complaints about blasting, others rarely receive complaints. People differ greatly in their response to blasting. Blasts which may be barely noticed by some individuals may be very troublesome to others.

Blasting noise generally increases with the amount of explosive, with atmospheric conditions and, with proximity to the blast. The area in front of a blast receives more noise than the

area behind it. The apparent blasting noise is greatest when there is least background noise. Blast noise tends to be greater when less explosive energy is absorbed into rock; large production blasts are often not as noisy as much smaller blasts used to break large boulders. In the later case, much of the energy goes into the air as noise.

Complaints about noises associated with excavating, transporting and processing of aggregate are less than blasting because such operations are either in remote areas or are rarely noticeable over the normal daytime noises. Trucks transporting rock from operations are objectionable to some people, particularly when they haul during quiet hours. The difference between the noise from blasting and that from various mining, processing and construction equipment is duration. Because of their very short duration, the greater noise levels from blasting can be tolerated without damage to hearing. The magnitude of noise is inversely proportional to the square of the distance from the noise to the source.

Existing Noise Levels

Four of the six potential aggregate sources have existing operations: the Old Cool Quarry, Chevreaux, Mississippi Bar, and Yuba River Dredge Fields. Aside from traffic on nearby roads and occasional aircraft, the major source of noise in the immediate vicinity of these operations are the operations themselves. Except for Mississippi Bar, none of the alternatives are located in noise-sensitive areas.

For the existing operation at Old Cool Quarry, blast noises at the point of nearest habitation are typically less than 135 decibels (Office of Surface Mining, 1979). Because of the current low production levels, the sound levels are much less. However, occasional louder blasts may occur. Truck and miscellaneous plant noises are generally much less perceptible at the nearest habitation, although sound levels may exceed 85 decibels at the source. The nearest residence is 2,000 feet from Old Cool Quarry; 3,000 feet from the Cool Quarry Amphibolite. Similar distances separate operations at Mississippi Bar and Chevreaux. For the Middle Fork bars and the Yuba River Dredge fields, the nearest residence is over a mile.

Project-Generated Noise

Mining and Processing Noise. During the mining operation at any of the sources, a number of noise-generating sources will be in operation. Some of the sources will be intermittent and some constant; some sources will be stationary while others will be mobile.

Major sources of noise generation will be drilling rigs, blasting, crushing, loading and hauling of equipment. Drilling and blasting will only occur at the quarry operations (Old Cool Quarry, Cool Quarry Amphibolite and Chevreaux). Noise generation can be expected to occur during nighttime hours due to high production rates necessitated by the construction schedule (2-3 year construction period). Table 4.7-1 details potential noise sources.

TABLE 4.7-1
DAYTIME SOURCES OF NOISE GENERATION

Noise Source	Maximum Anticipated Noise Level (4)
Blasting	130 dB (1)
Drill, 6"	89 dB (1)
Drill, 2-3", airtrack with compressor	83 dB (1)
Shovel (Cat 245)	75 dB (1)
Cone Crusher	79 dB (2)
Jaw Crusher	82 dB (2)
Screens	76 dB (2)
Dozer	80 dB (1)
Grader	80 dB (1)
Loader	79 dB (3)
Truck	80 dB (3)
Truck (Cat 733 B)	77 dB (1)
Scraper	81 dB (1)

(1) Deem 1985-1988
(2) Skega 1977
(3) USEPA 1971
(4) All levels A-weighted with slow meter response at 50 feet except blasting, which are linear, peak, at 1,000 feet.

Maximum noise levels at a given location can be estimated through the use of mathematical modeling which includes factors for attenuation due to atmospheric absorption, barriers topography, and distance. It is important to note that decibel measurements are not directly additive. For example, if the background noise level is measured at 50 dB and a 50 dB noise source is introduced (which is a doubling of noise pressure), the resultant noise level would be 53 dB, not 100 dB.

Transportation Noise. Changes in ambient noise levels would also derive from the transport of aggregate material to its point of use. The particular source dictates the type of noise impact. Transportation noise associated with aggregate sources within the Middle Fork canyon could either be from rail, conveyors or trucks. Noise generated by these sources would be confined to narrow corridors along the river. Outside the canyon, aggregate would be moved by large numbers of either trucks or trains. Because of the large quantity of material transported over a relatively short duration, temporary noise increases would be expected to occur.

4.7.2 IMPACT ANALYSIS

Noise impacts are significant if there is a substantial increase in ambient noise levels for adjoining areas. Construction and mining activities, especially blasting and operation of heavy equipment, would create temporary noise increases near the individual aggregate sources. Mining and construction noise impacts in the vicinity of each alternative would vary markedly because of the nature of the operations and because the noise strength of construction, mining and processing equipment ranges widely as a function of the equipment used and its activity level. Implementation of any of the potential sources near the dam site (Old Cool Quarry, Cool Quarry Amphibolite and the Middle Fork bars), would result initially in noises dominated by earth-moving equipment necessary to build roads and strip overburden. Later the noise environment would be dominated by mining, processing and transporting equipment. These impacts are considered significant and unavoidable (Class I).

Noise from blasting at the quarries would be the loudest, ranging up to 130 decibels 1,000 feet from the source. Earth-moving sources are generally the next loudest with equipment noise ranging from about 70-90 d(A) 50 feet from the source (Table 4.7-1). The greater noise produced by blasting is more tolerable because of its short duration. These blasting-related impacts are considered significant but mitigable (Class II). Noise from mining and construction equipment is more noticeable because of the extended generation intervals.

Spherically-radiating point sources of noise emissions are atmospherically attenuated by a factor of 6 dB per doubling of distance. The quieter earth-moving and mining noise sources would, therefore, drop below 60 dB by about 300 feet from the source while the loudest sources may still be easily detectable above the local background noise beyond 1,000 feet from operational areas such as processing facilities and quarry faces.

Aggregate handling and processing and small stationary noise sources have lower initial noise levels, so their corresponding noise impact zones are, therefore, much smaller. Noise emissions from haul trucks, compressors, pumps, etc., are generally attenuated to acceptable levels within 500 feet of the noise source. Smaller, discrete sources such as generators and compressors are also more readily controlled with heavy-duty mufflers designed to minimize noise generation. Their mobility and small size allow for their placement in areas where structures, walls or other barriers can shield sensitive receptors.

Temporarily increased noise levels can be anticipated from the transport of material to the dam site. Because they are powered by electricity, conveyors would have the least impact, followed by rail and trucks. Individual sources have varying proximities to sensitive receptors. Mississippi Bar and the Chevreaux property, because of existing residential development along rail and truck routes to and from these sources, would probably have the greatest potential for impact. Truck transport of material from the Yuba River dredge fields would also increase ambient noise levels along State Routes 65 and 193. These types of transportation-related noise impacts could be considered significant adverse impacts which cannot be avoided (Class I).

4.7.3 MITIGATION MEASURES

Mitigation for permanent aggregate operations normally includes limitations on mining and processing hours. In addition, quarries are usually limited with respect to the time of day when blasting may take place. Existing operations such as the Old Cool Quarry, Mississippi Bar, Chevreaux, and Western Aggregate at the Yuba River dredge fields have designated hours of operation.

However, construction scheduling will necessitate nearly continuous mining, processing and transporting of aggregate thereby rendering conventional mitigation ineffective. For this reason, conventional mitigation of noise impacts related to the various alternative aggregate sources would not be readily applicable. The increased activity at any of the sources would create significant adverse unavoidable short-term noise impacts to nearby sensitive receptors. This is particularly true for those sources that would utilize populated haul routes, such as Mississippi Bar, Chevreaux and the Yuba River dredge fields.

Efficient use of explosives can minimize blasting noise. Blast noise tends to be greater when less explosive energy is absorbed into rock; large production blasts are often not as noisy as much smaller blasts used to fragment large boulders. Effective mufflers can be used to minimize noise from earth-moving equipment and other stationary noise sources such as generators. Also, the relative remoteness and the lack of sensitive receptors in the vicinity of the three sources near the dam site serves to minimize noise-related impacts.

4.8 RECREATION

4.8.1 EXISTING CONDITIONS

Middle Fork Sand and Gravel Deposits

The Middle Fork of the American River is generally characterized by rushing rapids, deep clear pools and steep canyons surrounded by wooded ridgelines. This setting allows for a diversity of unique recreation opportunities from whitewater rafting, swimming and fishing to picnicking, recreational gold mining and hiking. In addition, the canyon's diverse flora, fauna and geological features provide excellent conditions for passive enjoyment of its natural features.

The United States Bureau of Reclamation contracted with the Department of Parks and Recreation (CDPR) to provide recreation and public-use management services on the lands within the multipurpose Auburn Dam project boundaries, known as the Auburn State Recreation Area. This area includes 42,000 acres and 48 miles of the North and Middle Fork of the American River from the dam site to Iowa Hill bridge and Oxbow Reservoir, respectively.

The area's proximity to major population centers and diverse recreation base make the Auburn State Recreation Area (Auburn SRA) one of the most used and significant recreation resources in northern California (NRA Feasibility Report). The expected growth of the surrounding Mother Lode and Sacramento Metropolitan areas will make this resource even more important to future generations. The recreation area is especially accessible to the surrounding populations because of its location near major transportation corridors. The area is a 2-hour drive on Interstate 80 from much of the San Francisco Bay Area, and even closer from Reno. State Highway 49 traverses the Auburn State Recreation Area from the north and south.

Local interest in outdoor recreation is intense in the Middle Fork canyon. Based on 1988 Attendance Data provided by CDPR, the confluence area supports approximately 23% of the annual visitation in the Auburn SRA. Other significant Middle Fork recreational areas include Mammoth Bar, which supports 8% of the annual visitation, and Cherokee Bar supporting approximately 2% of the annual visitation. Although current attendance data was unavailable, recent trends show increasing demands for equestrian, hiking and biking trails, and a continued deficit in resources to meet this demand.

Equestrian, hiking and mountain biking trails exist throughout the Middle and the North Fork of the American River. Some trails have historical significance such as the Western States Trail which was originally used by the Paiute and Washoe Indians and later early pioneers and miners. It is now the route of two international endurance races: the Tevis Cup Ride for horses and the Western States 100-Mile Endurance Run (on foot). Throughout most of the year, several other significant athletic events occur on this trail as

well. Other trails in the project area are utilized by off-road vehicle and mountain bike enthusiast.

Both the North and Middle Fork of the American River are popular rivers for whitewater recreation. CDPR, manager of whitewater recreation in the basin, has witnessed a steady increase since 1979 in commercial river rafting and kayaking, particularly on the Middle Fork.

The Middle Fork, in general, is technically less challenging than the North Fork. It offers 24 miles of class I to III river with some opportunities for advanced whitewater (Class IV to VI), such as Mammoth Bar. The upper Middle Fork, from Oxbow Reservoir to Ruck-a-Chucky is the more challenging portion of the river. Ruck-a-Chucky to Mammoth Bar provides suitable water (Class II, portage at Mammoth Bar) for less experienced river rafters, canoeists, and families with small children.

Old Cool Quarry (Spreckles)

The Old Cool Quarry is located on the south side of the Middle Fork of the American River, within the Auburn SRA and approximately one mile north of the town of Cool. The quarry is an existing mining site adjacent to the Auburn Lake Trails subdivision. It is visible from Highway 49 and Foresthill Road on the north side of the Middle Fork, from Auburn Lake Trails subdivision, and various locations along the river. Mining operations are performed on the higher elevations, above the canyon floor. Access to the site is limited and there are no specific recreational uses that occur within the property. The lower portions of the site may be utilized for hiking. Recreational opportunities within the vicinity of Old Cool Quarry are related to those described for the above Middle Fork Sand and Gravel Deposit discussion.

Cool Quarry Amphibolite

The Cool Quarry Amphibolite is adjacent and directly downstream from the Old Cool Quarry. The project site contains rugged terrain and there are no recognized recreational activities that occur on-site with the exception of possible hiking trails near the Middle Fork. Recreational opportunities within the vicinity of the Cool Quarry Amphibolite are related to those described in the Middle Fork Sand and Gravel Deposit discussion.

Chevreaux Quarry

The Chevreaux Quarry is located in Placer County and is approximately 2 miles north of the town of Meadow Vista on Lake Combie. Residences are located adjacent and to south and east of the site. Recreational facilities in the nearby community of Meadow Vista include local parks and school playing fields. Lake Combie provides water oriented recreation, such as boating and fishing, to the immediate vicinity. No state or locally sponsored recreational facilities exists at the lake as it is primarily a holding reservoir for agricultural irrigation and

is managed by the Nevada County Irrigation District. According to Ed Nuehart (NCID), boating facilities on the lake are allowed by NCID, but are privately owned by local residents.

Yuba River Dredge Fields

The Yuba River resource area extends along the Yuba River, upstream from the town of Marysville. This 8,500-acre area has a long history of hydraulic mining, dredging for gold and aggregate. The surrounding area is predominantly agriculture, with scattered rural residences.

Existing recreational activity associated with the area may include local fishing in certain areas outside of the vicinity of mining operations. Other recreational activities in the area would be independent of local or state designated parks or recreations areas and may include river-oriented recreation activities.

Mississippi Bar

Mississippi Bar is located on 160 acres of federally-administered land and is approximately one mile upstream of Nimbus Dam and approximately 1,000 feet from the shoreline of Lake Natoma. It is surrounded by State of California lands, except for the eastern boundary of the property where federal lands are administered by the Bureau of Reclamation.

Lake Natoma is a high use area for many recreational activities such as boating, swimming, sailing, and fishing. In addition, the heavily used American River Parkway provides bicycle and equestrian trails through a portion of Mississippi Bar which has been reclaimed and turned over to CDPR. Other reclaimed areas allow for canoeing and fishing.

The General Plan for the Auburn Reservoir Project/Folsom Lake State Recreation Area (California Parks and Recreation, 1980) suggested that possible recreational facilities that the CDPR could develop at Mississippi Bar could include dredging to create a new landscape/use area, shallow lagoons and channels for canoeing, swimming, hiking as well as an entrance gate and parking lot. Some areas have already been reclaimed and turned over to the State. However, the reclaimed areas result in undulating finished grades covered with river-rock cobblestone which are not suitable for most recreational activities.

4.8.2 IMPACT ANALYSIS

Middle Fork Sand and Gravel Deposits

Aggregate mining on the Middle Fork would require establishment of an aggregate extraction, conveyance and processing operation along the Middle Fork of the American

River from Cherokee Bar to the proposed dam site. For a more detailed description of this project alternative, please refer to the Project Description in Section 3.0.

Aggregate mining would eliminate ten bars between and including Mammoth Bar up to Cherokee Bar. Stream morphology, trails, and dirt roads providing public access to the river would be unavoidably and significantly impacted by this alternative, thereby causing significant impacts to current recreational activities in the Middle Fork Canyon. The following discussion further defines these significant impacts to recreation associated with the Middle Fork of the American River.

Aggregate mining of the Middle Fork Bars would significantly alter the stream morphology of the river. As discussed in Section 4.10, In-Stream Impacts, "Ultimately the stream would likely form a boulder-strewn bedrock channel, similar to Ruck-a-Chucky Rapids upstream from Cherokee Bar." It is predicted that the removal of the Middle Fork bars would leave deep pools, with boulders along the sides of the river channel; therefore, it is not known whether or not this portion of the river would still offer Class I or Class II rapids. In addition, another scenario suggests that an increase in mandatory portages caused by large drops in the river channel could render this portion of the Middle Fork unrunnable for rafting, canoeing and swimming for two reasons. First, the removal of the bars would decrease the number of suitable areas to "take-out" of the river; and second, there would not be as many areas to retrieve rafters.

Mammoth Bar supports approximately 40,000 user days per year, or 8 percent of the total Auburn State Recreation Area use (CDPR). The removal of Mammoth Bar use area would result in an unavoidable long-term significant impact to the recreational opportunities this area supports. The off-road vehicle roads and picnicking opportunity would be permanently altered. Due to the uncertainty of what form the river would take after removal of the sand and gravel bars, swimming and rafting would also be impacted.

Other bars upstream support overnight camping and day use activities associated with river recreation, such as swimming, hiking, horseback riding, fishing, and gold dredging. The cumulative loss of these bars combined with the loss of Mammoth Bar would result in a significant unavoidable impact to the recreation value on this portion of the river. Mitigation of impacts to recreation would require development of new recreational staging areas that would support those activities that were affected, such as camping areas, equestrian and hiking trails, picnic areas, and off-road vehicle roads. However, the character of the current recreational experience could not be recreated.

Construction of the conveyor system and maintenance roads would require extensive grading and excavation of vegetation, in order to provide a 20-foot right-of-way. Public access and all recreation activities associated with the river would be restricted during construction of the conveyors and the maintenance roads until excavation of the aggregate was completed.

Special events, such as the Tevis Cup and the Western State 100-Mile Endurance Run, would either have to be rerouted around these closed areas during this period or mining operations could be shut down on schedule race days. Public access would be controlled in areas of operation which would include the conveyors, maintenance roads, and mining areas. All recreational activities would be restricted in areas being excavated and downstream along the conveyor system to the dam site. This would be considered a short-term significant unavoidable impact (Class I).

In addition, the conveyor system would carry material past the confluence to the proposed dam site. The confluence area is one of the most popular picnicking, swimming and fishing areas in the canyon. Public access and parking are provided off of Highway 49, approximately three miles east of Auburn. This area supports 23 percent of the total Auburn State Recreation Area (CDPR). Recreational activities would be restricted for approximately the same period of time as upstream restrictions. This would constitute a short-term significant unavoidable impact (Class I) as mining operations would close this area to recreation. Following completion of this project, the confluence area would retain the same recreational benefits as it does today, although stream morphology could be slightly altered as the river seeks out equilibrium following removal of the upstream sand and gravel bars.

Transportation routes for delivery of material to the proposed dam site would temporarily disrupt portions of existing trails throughout the period of mining operations. This would be a short-term significant impact to trail users; however, once mining operations were completed, trails could be opened to the public.

Once mining operations were completed, equipment would be dismantled and removed from the canyon. The remaining features would result in a significantly different Middle Fork River Canyon. Changes to the river channel would be unavoidable and would result in permanent significant impacts to rafting. In addition, portions of equestrian, hiking, biking and ORV trails would be permanently altered, and camping and day use areas in this portion of the river would be reduced. Significant visual impacts (see Section 4.9) would alter the scenic attributes of the canyon which enhance the recreational experience.

Old Cool Quarry (Spreckles)

The Cool Quarry has no known recreational activity associated with the site. However, Old Quarry Road, which is also a portion of the Western States Trail, would be temporarily closed for public use until conveying operations were completed. This would be a short-term significant adverse impact (Class II) to trail users and special athletic events that use this trail. Rerouting, or closure of mining operations could be arranged to mitigate this impact to a less than significant level.

Cool Quarry Amphibolite

Cool Quarry Amphibolite has the same recreational influences as those associated with the Old Cool Quarry, described above.

Chevreaux Quarry

Bear River and Chevreaux Quarry is located on the northern section of Lake Combie. Due to its relatively isolated location on the lake, an increase in quarry operations would not have a significant impact to recreation activity.

Yuba River Dredge Fields

The Yuba River Dredge Fields are not associated with any significant recreational activities. However, the 8,500-acre resource area may have illegal activity such as off-road vehicles or hunting. Therefore, increased mining operations on this site would not have a significant impact on legal recreation activities, although some disruption of unauthorized activities could occur.

Mississippi Bar

The existing bike trail is located in areas that have already been reclaimed and turned over to the state. The increase in truck traffic would impact equestrian trails located at the entrance to this facility which would constitute a significant adverse impact to trail users (Class II). Mitigation would require trails to be temporarily rerouted along the bike trail in the American River Parkway.

4.8.3 MITIGATION MEASURES

Unavoidable significant adverse impacts to the Middle Fork could not be mitigated to a less than significant level as recreation and scenic values in this area would be permanently altered. Impacts associated with other alternatives would be mitigated to a less than significant level. After mining operations were completed, no residual impacts would remain at site alternatives, with the exception of the Middle Fork.

Middle Fork Bars

Most impacts caused by sand and gravel mining operations associated with the Middle Fork bars would be unavoidable and long-term since recreation opportunities and scenic values would be significantly altered. Partial mitigation would require development of new staging areas for camping, picnicking, public access areas, and reclamation of damaged trails. Development of these features could not be fully mitigated since present recreational activities would not be replaced. In addition, scars left by the conveyor alignment could be

partially mitigated through revegetation efforts; however, full recovery would not be possible.

- Require development of new staging areas for impacted activities, such as picnicking, ORV trails, camping, and parking.

- Require revegetation in disturbed areas, such as the conveyor and maintenance road alignment and portions of excavated bars that would sufficiently support such efforts.

Old Cool Quarry (Spreckles)

Impacts to the Western States Trail would be short-term but significant. Rerouting the trail around the quarry during the period of operation would allow for continued trail use. After dismantling the conveyor system, any trail damage caused by the conveyor shall be repaired.

Cool Quarry Amphibolite

Mitigation would be the same as described above.

Chevreaux Quarry

No significant impacts have been identified; therefore, no mitigation would be required.

Yuba River Dredge Fields

No significant impacts have been identified; therefore, no mitigation would be required.

Mississippi Bar

Impacts to the equestrian trails located at the entrance of the mining operations would be mitigated by temporarily rerouting the trail into the American River Parkway.

4.9 VISUAL RESOURCES

4.9.1 EXISTING CONDITIONS

Middle Fork Sand and Gravel Deposits

The Middle Fork of the American River flows at the bottom of a steep rugged canyon densely vegetated with chaparral, ponderosa pines and a variety of oak trees. The river flow is accentuated with rushing white water rapids dispersed with deep clear pools. The canyon's diverse flora, fauna, and geological features provide the aesthetic setting for recreation and nature enthusiasts.

Historic trails, bridges and structures exist throughout the Upper American River canyons, communicating a connection to the past as well as a sense of the canyon's importance to the region. The Western State Trail occupies the Middle Fork and was the original trans-Sierra crossing, from Auburn to Squaw Valley, and a lifeline for high country pioneers and gold miners in the 19th and early 20th century. Presently, the trail has been included on the National Register for historic sites. It is also a popular route for several horse and foot endurance events, which use this trail in the spirit of its historical importance. The popularity of these events have increased over time and is primarily due to the physical challenge, beauty, and history the canyon exudes.

Other significant features include No Hands Bridge, a reinforced concrete span bridge which was the largest constructed in its period; and Grizzly Bear House on Foresthill Road which was a major way station for pioneers. These historic features enhance the aesthetic value of the canyon by providing a visual connection to the past.

Widenings in the river canyon bottom currently provide staging areas for picnicking, fishing, and camping. The Middle Fork bars and drainages of canyon slopes provide the canyon with bands of varying shades of trees and shrubs associated with riparian zones. These bands that form along the river's edge are occasionally replaced by smaller shrubs and grasses which allow for a view of gently rolling topography at the base of steep slopes and rugged terrain. The color, textures and lines created by the riparian zones are communicated through the changes in the direction of the river flow, vegetation and slope. It is these attributes, combined with the historical importance of the canyon which influence the aesthetic value and visual interest of the entire canyon.

Old Cool Quarry (Spreckles)

Old Cool Quarry is located on the south side of the Middle Fork, upstream from the confluence and the Highway 49 bridge. The aesthetic and visual influence is the same as described for the Middle Fork Bars. Prominent views of the quarry can be seen from Highway 49 on the north and south side of the Middle Fork, Foresthill Road on the north side, residences within the Auburn Lake Trails subdivision, and from the river canyon floor.

Significant views of the site from the river begin approximately at Kennebeck Bar and continue downstream to Mammoth Bar.

The quarry site has been stripped of all vegetation, accentuating the presence of heavy mining operations. Heavy equipment and trucks operate on the terraced portions of the ridge, creating fugitive dust clouds. The flattened grade of the mining operation, void of vegetation, contrasts sharply with the steep densely vegetated slopes of the canyon. Figure 4.9-1 shows existing operations as seen from the Auburn Lake Trails subdivision.

Cool Quarry Amphibolite

The proposed Cool Quarry Amphibolite site is located west of the Old Cool Quarry and is still in its natural state. Figure 4.9-2 shows the site in relation to the Old Cool Quarry. This site is characteristic of typical canyon terrain and vegetation and is influenced by the same aesthetic and visual interests. The Highway 49 bridge and the confluence area has a direct view of the site. Additionally, Highway 49 passes the site on the south side of the canyon with a direct view, as well as the north side of the canyon at various locations between the confluence and Mammoth Bar. The site is not within view of residences in the Auburn Lake Trails subdivision.

Chevreaux Quarry

The Chevreaux Quarry is approximately 2 miles outside of the community of Meadow Vista. The community is a rural town with large residential parcels varying between 10 and 20 acres. The quarry site is located on the northern end of Lake Combie where the Bear River feeds into the lake. This lake has a narrow, elongated shape which is oriented to the north and south. Tree-covered hills surround the lake, and provide lakeside residents with an aesthetically pleasing environment. Numerous residences are located on both the west and east sides of the lake, towards the southern end. Boat docks are found primarily on the east side.

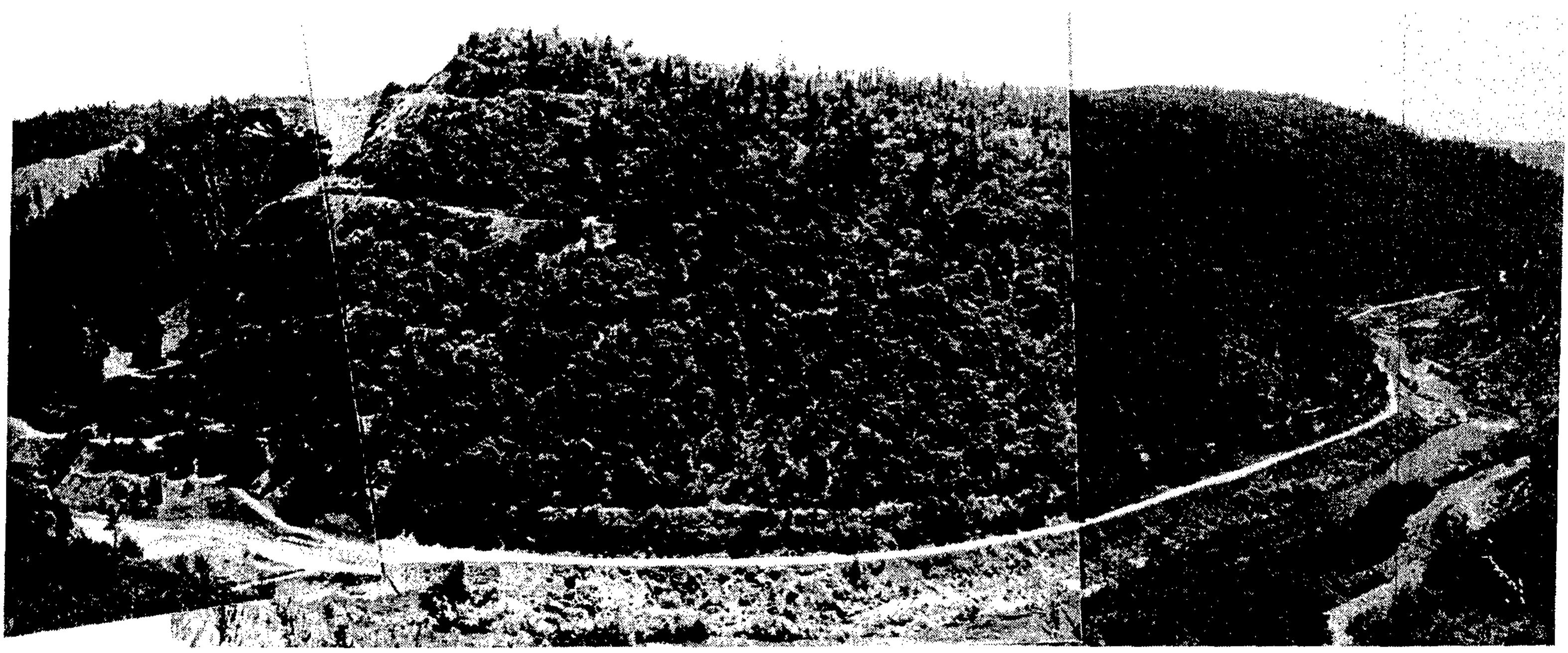
The quarry site is hidden behind a ridge oriented east/west which blocks views of the operation from lake residents. Some portions of the quarry site may be in view of residences within the vicinity of the site. The operation includes typical mining equipment such as conveyors, screening processes, heavy equipment and stockpiling areas. In addition, the quarry operator has rights to dredge sand from the entire lake; however, dredging is practiced primarily in the northern portion of the lake, away from most recreational activity. Figure 4.9-3 is a topographic map showing the orientation of the quarry site in relation to lake residences and the Meadow Vista community.

Yuba River Dredge Fields

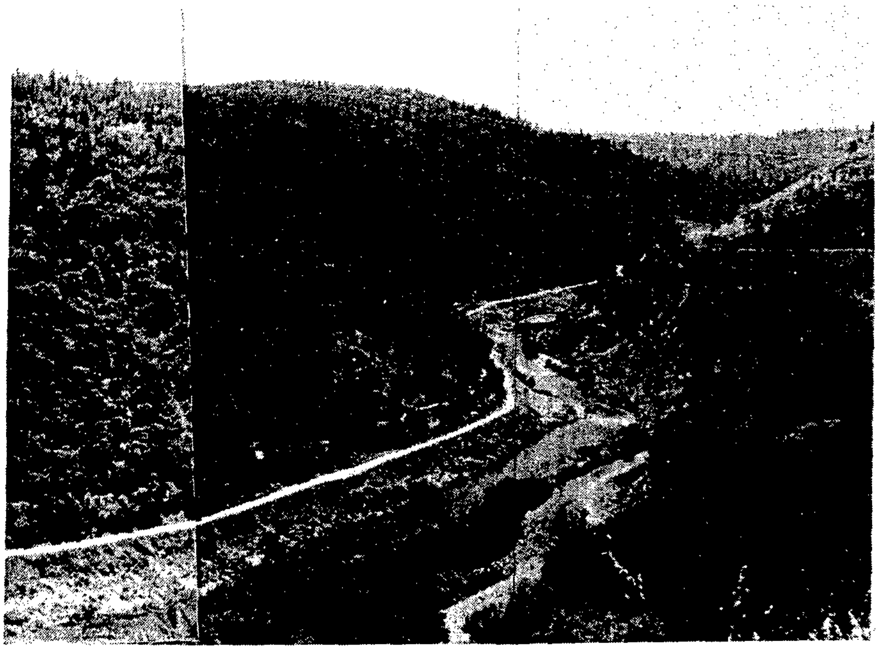
The Yuba River Dredge Fields consist of 8,500 acres of sand and gravel resources and are adjacent to agricultural lands. Some rural residences exist; however, they are distant. The

Views of Cool Quarry from the Auburn Lake Trails Subdivision

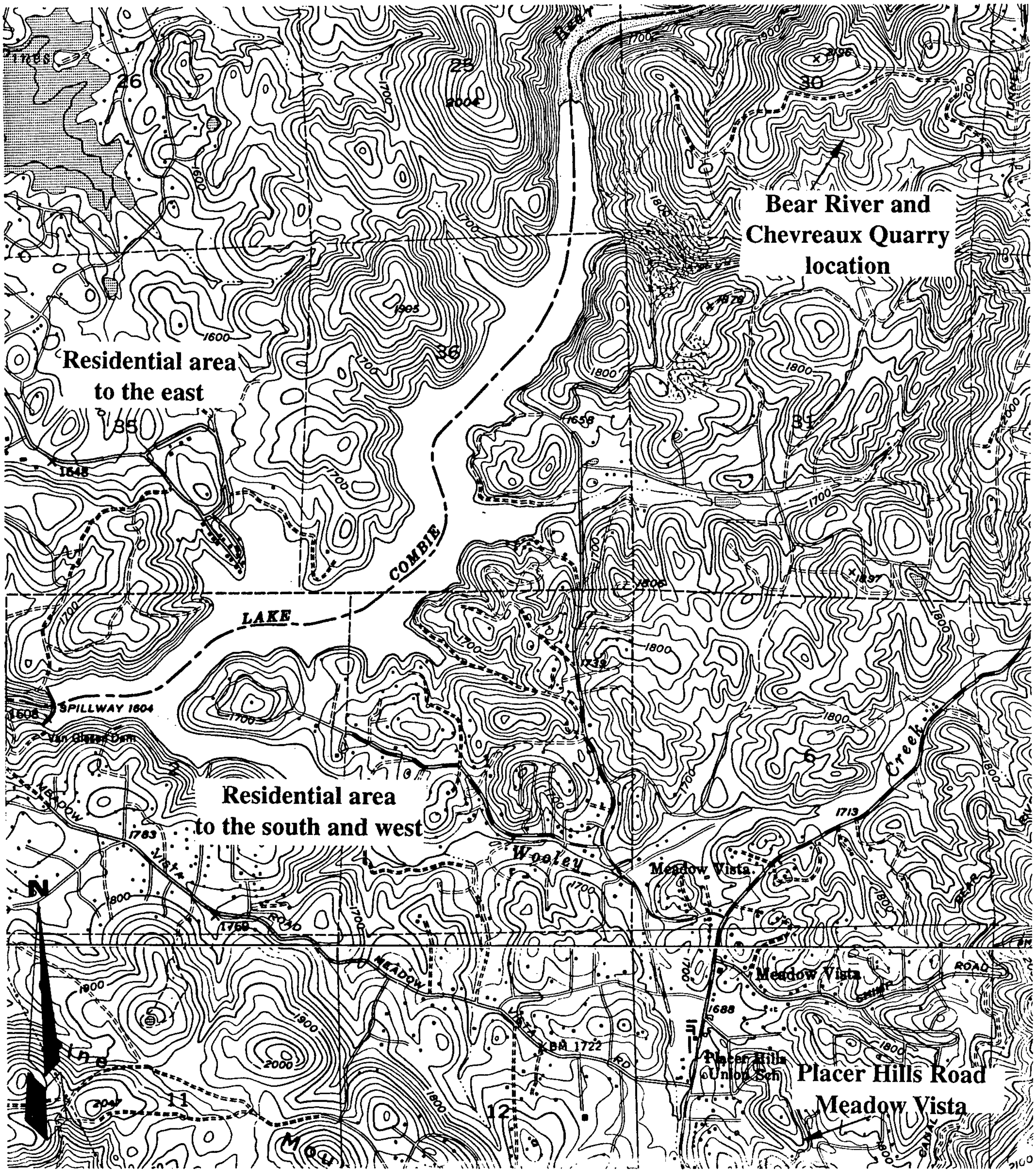
View of Old Cool Quarry an



View of Old Cool Quarry and proposed Cool Quarry Amphibolite



Quarry Amphibolite



Land Uses in the Chevreaux Quarry Area

mining operations are in the vicinity of the Yuba River; however, the river is out of view from most mining activities. Surrounding views include very distant foothills and occasional tree groupings which are primarily associated with the rural residences. Large piles of sand and aggregate create an unnatural environment with little vegetation. Most of the equipment related to the mining activities are not noticeable from local roads.

Mississippi Bar

Mississippi Bar is located adjacent to and on the north side of Lake Natoma. It is surrounded by tall trees such as oaks, willows, alders; and small shrubs such as blackberry, elderberry and coffeeberry. The American River Parkway bicycle trail divides an area which has been reclaimed and turned over to the State. Large equipment is not visually prominent; however, reclaimed areas are covered with river cobblestones with a finished undulating grade. There is little vegetation in the majority of these reclaimed areas, with the exception of windrows and vegetated "islands" which were preserved for endangered species. Residential homes exist to the north on top of bluffs, overlooking the lake and the excavation site. Direct views of the site are minimal from Main Avenue and other residential streets within the vicinity.

4.9.2 IMPACT ANALYSIS

The visual impacts associated with the extraction of aggregate for the proposed dam site are most significant when considering the Middle Fork bars and the Cool Quarry Amphibolite alternatives. The other alternatives, already in operation, would not have significant impacts to visual resources in their surrounding area, except possibly the increased amount of fugitive dust generated by increased operations. A more detailed discussion of project alternative impacts follows.

Middle Fork Sand and Gravel Deposits

Sand and gravel deposit operations in the Middle Fork would extract sand and gravel from 10 bars (Mammoth Bar upstream to Cherokee Bar) and transport the aggregate by conveyor approximately 12 miles downstream to the proposed dam site, as described in Section 3.0, Project Description. After the excavation, the bars would be permanently lost and a highly disturbed appearance would be present in the river channel. Also, scars from the excavation of the right-of-way for the conveyor would follow the north side of the canyon from Cherokee Bar to the proposed dam site. The unavoidable long-term change in the visual character of the canyon would be a significant adverse unmitigable impact to the Middle Fork (Class I).

The bars provide a visual transition between the canyon walls and river. Riparian zones on the bars provide a band of deciduous plants and trees which provide a "bench" for recreation. Significant amounts of riparian vegetation would be excavated. Over time, a new

band of vegetation would regenerate. However, it is impossible to determine how well the new stream channel would regenerate new vegetation as the bars would be mined to bedrock below, thus leaving a minimal amount of suitable topsoil available for regeneration. This scenario assumes regeneration would be minimal in the long term, thereby permanently altering the visual character and aesthetic essence of the Middle Fork. The alteration of these visual amenities would be an unavoidable significant adverse and unmitigable impact to the Middle Fork canyon (Class I).

As mentioned above, the conveyor system would be constructed along the north side of the river. The proposed alignment for the conveyor system is shown in Figure 3-21. The proposed alignment for the conveyor would require a right-of-way approximately 20 feet wide. This would require excavation of existing vegetation. Extensive grading would require cuts into the canyon slopes. The grading cuts would be visually prominent for the 12 miles stretch from Cherokee Bar to the dam site. Once mining operations stop, the right-of-way would not recover to a natural state. Some areas of the alignment would revegetate over time. However, the cuts in the slopes would never fully recover back to a natural state and would become a permanent visual feature in the canyon. This would create a long-term significant impact (Class I) which could only be partially mitigated through revegetation efforts.

Old Cool Quarry (Spreckles)

Old Cool Quarry is an existing mining operation and the visual character of the site has already been substantially altered from its original state. Use of this facility would require an increase of excavation and processing, which could increase the amount of fugitive dust. This could be considered a significant visual impact (Class II); however, it would be a short-term impact which could be mitigated.

Cool Quarry Amphibolite

This proposed site would excavate existing vegetation, called overburden, in order to get to the actual stone. Large cuts into the slope would be made in order to create a pad to stage operations. Use of this site would result in a permanent unavoidable significant adverse impact (Class I) to the visual quality of the canyon. Views of this site would be accentuated by its close proximity to the existing Cool Quarry. The combined operations would degrade the natural character of the canyon. Sensitive viewing locations that would be impacted (Highway 49 bridge, the north side of the canyon, and the confluence of the North Fork and Middle Fork rivers) would be directly impacted by enlarging the mining scar to include the site. The scenic value of the canyon would be damaged as this area is directly visible from the confluence, a heavily used recreational area, and the Highway 49 travel corridor.

Chevreaux Quarry

Chevreaux Quarry is an existing mining operation. Use of this facility could require increased dredging on Lake Combie, which could mean dredging further to the southern end of the lake toward residences. This would create an adverse visual impact (Class III) to existing residents; however, it would not be considered a significant impact as it would be short-term in nature and would not be a daily activity. Increased mining operations would not increase visual impacts beyond those currently associated with the site.

Yuba River Dredge Fields

Since the Yuba River Dredge Fields have a historical presence in the area and are out of view from major developed areas, increased mining would not be noticeable (except by the increase of truck activity in the area). No significant visual impacts would occur with implementation of this alternative.

Mississippi Bar

No significant visual impacts would occur by using this existing facility. Operations are currently well-hidden from residents and recreational users. Possible increases in fugitive dust and noise may occur but could be mitigated to a less than significant level.

4.9.3 MITIGATION MEASURES

Existing aggregate extraction sites would cause very few significant impacts. Visual impacts created by fugitive dust could easily be mitigated to a less-than-significant level by frequent watering of activity areas. Significant impacts caused by the Middle Fork Sand and Gravel Deposits and Cool Quarry Amphibolite would be unavoidable and unmitigable.

Middle Fork Sand and Gravel Deposits

The impacts associated with the excavation of the Middle Fork bars are unmitigable. Reclamation efforts to reestablish the visual character and aesthetic value unique to the bars would not be viable. Therefore, a statement of overriding considerations would have to be made.

- Require revegetation and establishment of native plant material within the alignment of the conveyor.

- Require reclamation of the canyon bottom to the extent possible.

Old Cool Quarry (Spreckles)

- Require watering trucks to keep fugitive dust to a minimum.
- Require revegetation and establishment of native plant material within the alignment of the conveyor.

Cool Quarry Amphibolite

The significant impacts caused by establishing quarry operations on this site would be permanent and unmitigable.

- Prepare reclamation plan prior to commencement of mining activities.

Chevreaux Quarry

- Require watering trucks to keep fugitive dust to a minimum.

Yuba River Dredge Fields

No mitigation is required.

Mississippi Bar

- Require watering trucks to keep fugitive dust to a minimum.
- Require effective mufflers on earth-moving equipment and generators.

4.10 IN-STREAM IMPACTS

The U.S. Army Corps of Engineers has identified ten gravel bars along the Middle Fork, American River near Auburn, California as one of a number of potential sources of construction material for the Auburn Dam. This section identifies potential hydraulic and geomorphological impacts to the river's stream channel and floodplain occurring due to sand and gravel mining along a 7-mile reach of the river starting 5 miles upstream from the dam site. Because hydraulic and geomorphic impacts are relevant only to floodplain mining of the gravel bars, discussion of other aggregate sources is excluded from this section of the report. These types of impacts may be attributed to other alternatives (Yuba River Dredge Fields and Mississippi Bar). However, these types of impacts are considered in other sections of this report through analysis of environmental issues that would be affected by geomorphic change.

The mining operation would occur over a 2-3 year period, excluding the rainy season, and would remove over 90 percent of the coarse alluvial material from the canyon bottom. Information in this section regarding current and past conditions within the floodplain and surrounding drainage basin was obtained from published USBR (U.S. Bureau of Reclamation), Corps (U.S. Army Corps of Engineers) and USGS (U.S. Geological Survey) reports. The discussion of potential impacts and resulting conclusions stem from recorded observation of past flooding events and qualitative application of basic stream hydraulics to flow data through the river reach of interest.

4.10.1 EXISTING CONDITIONS

Location and Description of the Deposits

The aggregate deposits are contained within a series of gravel bars located along a 7-mile section of the Middle Fork, starting approximately one mile upstream from the confluence of the Middle and North Forks (Figure 3-1). Mammoth Bar, the furthest downstream of the bars, is approximately 5 river miles from the proposed dam site. Cherokee Bar, the uppermost bar, is approximately 12 river miles from the dam site.

The Middle Fork bars were investigated by the USBR during preliminary engineering studies for the original multi-purpose project (USBR, 1968; 1976). The deposits are a series of lateral and point bars which contain an estimated 8.6 million cubic yards of sand and gravel evenly distributed between the 10 bars. Table 4.10-1 summarizes quantity and compositional information on each bar. The bars consist of sand, gravel and finer detritus (silts and clays) as well as cobbles and boulders eroded from the Middle Fork drainage basin (see Section 3.0). Results of a sampling study conducted by the U.S. Bureau of Reclamation indicate that the bars consist on average of 46 percent gravel, 36 percent sand, 8 percent fines and 12 percent particles larger than 3 inches (cobbles and boulders). There does not appear to be any systematic variation in the distribution of particle sizes with depth in the bars or between bars along the Middle Fork (Corps, 1991). This may be due to

TABLE 4.10-1
SUMMARY TABLE OF MIDDLE FORK AMERICAN RIVER
SAND AND GRAVEL DEPOSITS

BAR NAME	SIZE (SQ.FT)	DEPTH* (FT.)	PROJECTED YIELD (CU.FT.)	% > 3 in. **	% SAND	% FINES
Mammoth	971,250	31.4	1,129,500	14	36	9.5
Texas	996,000	31.2	1,150,900	12	34.5	8.5
Browns	599,000	30.4	674,400	9	36	7.4
Kennebeck	719,000	31.5	839,000	6	51	6
Hoosier	649,000	25.9	622,500	8	40	6.7
Buckeye	1,104,000	27.6	1,128,000	15	32	7.3
Maine	249,000	21.0	194,000	13	15	7.1
Philadelphia	907,000	23.7	705,000	17	28	10.6
Poverty	1,152,000	27.5	1,173,000	10	52	5.8
Cherokee	1,484,000	18.0	989,000	18	22	12.7
Exposed Bar Total	8,830,250		8,605,300	12	36	8.2
Average Depth		26.82				

* Depth to bedrock

** +6-inch-sized particles were obtained from within 38 inches diameter casing with the clamshell sampler.

deposition from upstream hydraulic mining activity, as well as later disturbance of the bars by dredging (Corps, 1991).

River Channel Morphology

During the nine to ten-month low flow season, flow in the river is dictated by controlled releases from the Oxbow Powerhouse located at Hell Hole Reservoir on the Rubicon River. During this period, flow is confined within an incised channel that traverses or circumvents individual lateral and point bars. Bank steepness ranges from nearly vertical on the convex side of river bends to gently sloping on the concave side. Bank heights range from in excess of 12 feet in cut bank areas to zero at the water surface of the point bars.

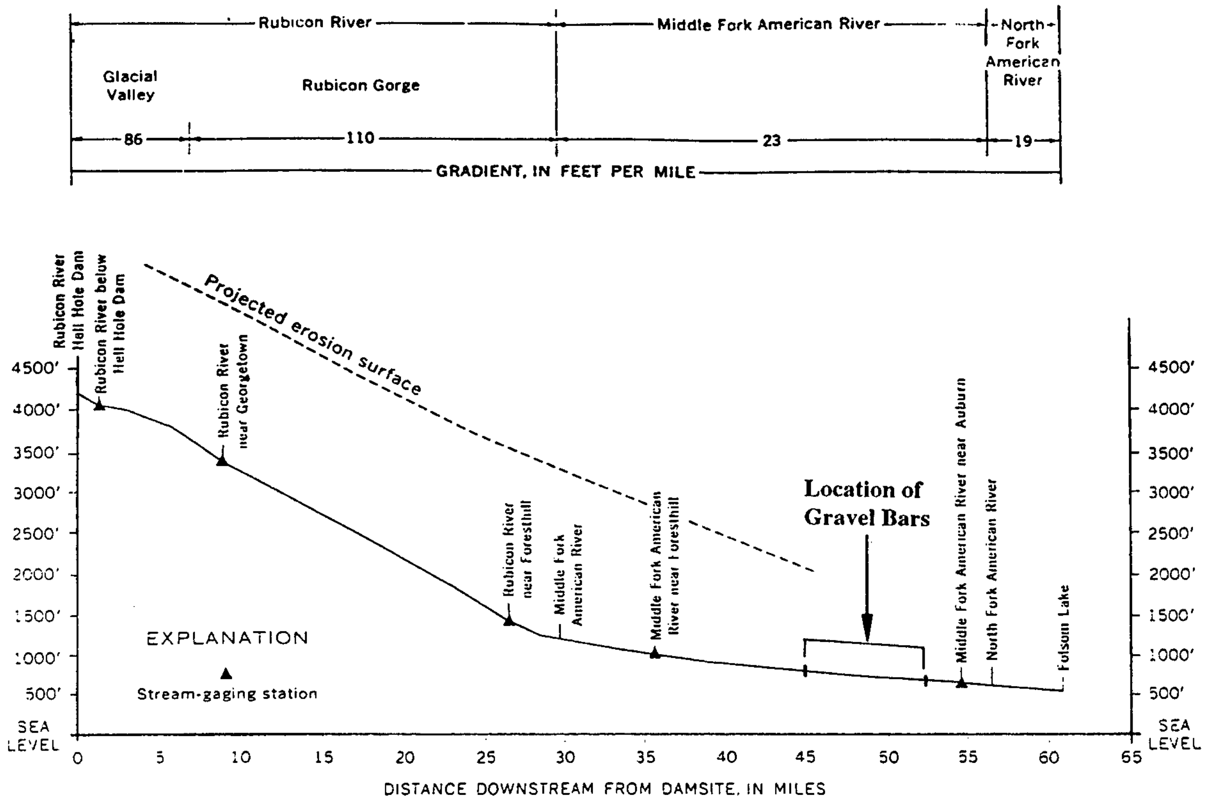
The accumulation of material along this reach of the river is the result of a declining gradient and transport capacity. Figure 4.10-1 shows the longitudinal channel profile from Hell Hole Dam to Folsom lake. Upstream from Cherokee Bar, the channel consists of boulder strewn bedrock surface (Ruck-A-Chucky Rapids) with little or no floodplain. Photographs taken in the 1850s (Turner, 1983) confirm that the bars were similar in extent previous to hydraulic mining activities.

In its present configuration, the river system in the vicinity of the gravel bars appears to have reached a state of equilibrium. Mature morphological features such as laterally accreted point bars and a deeply incised stream channel with vegetated banks, indicate that the river system has distributed any excess sediment loading created by upstream hydraulic mining activities. Figures 3-2 through 3-13 (see Section 3.0, Project Description) show large-scale aerial photos of the individual bars.

Based on the results of a seismic refraction survey conducted on the bars (USBR, 1968), depths to bedrock in the bars were generalized as follows: 1) Mammoth and Kennebeck 40-90 feet; 2) Texas Brown and Poverty 30-40 feet; 3) Hoosier, Buckeye, Philadelphia and Cherokee 10-20 feet. The seismic velocities through the sand and gravel were in the range of 1200-2500 fps (feet/second), which indicates the deposits are rather loose and unconsolidated. The exposed gravel bars along the Middle Fork were estimated to cover an area in excess of 206 acres, which could yield approximately 8.6 million cubic yards. Materials within the river channel could yield another 1.0 million cubic yards.

Drainage Basin

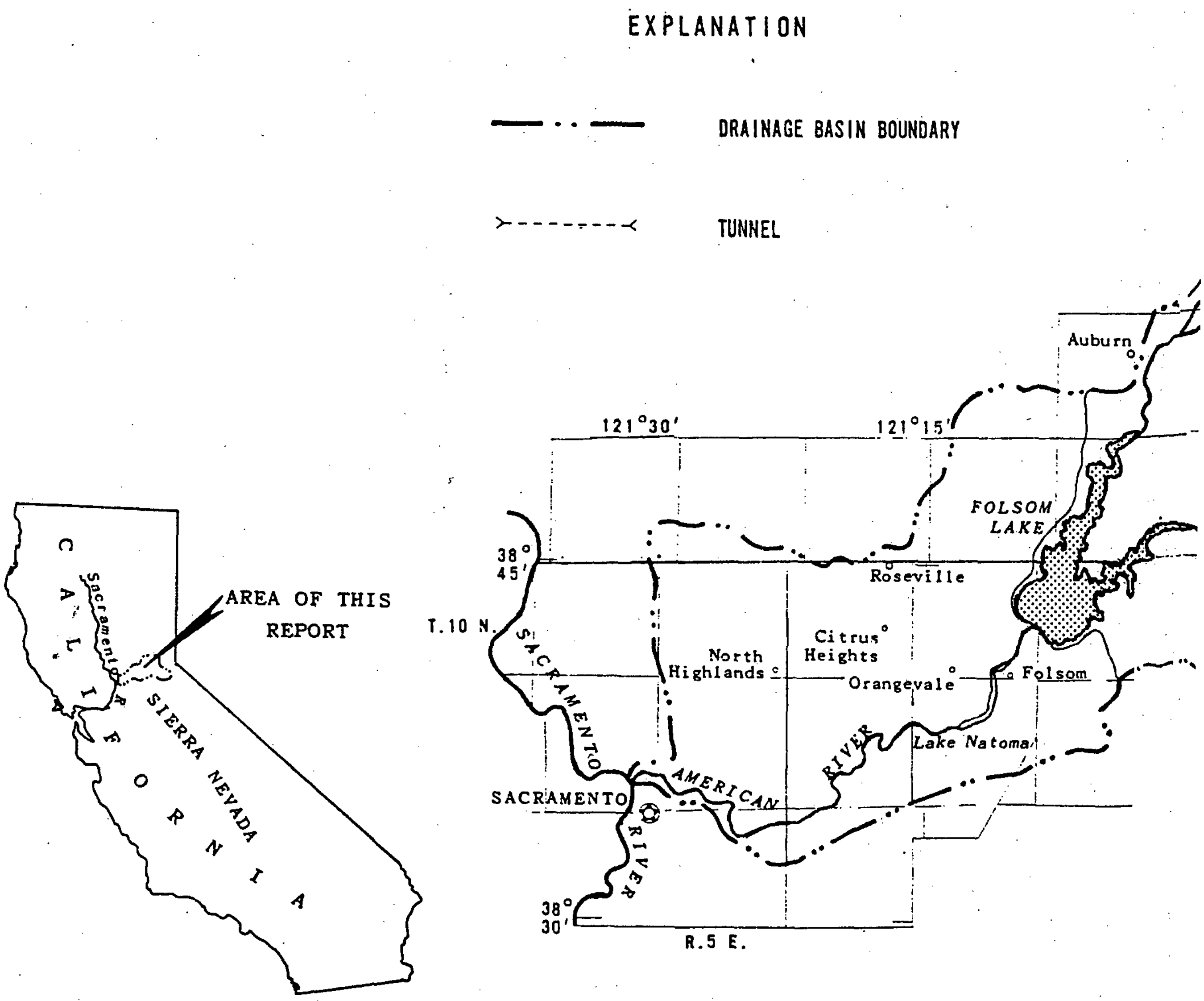
The total drainage area to the dam site is 974 square miles (Figure 4.10-2). There are three major streams within the basin, the North Fork American River, the Middle Fork American River, and the Rubicon River. Each stream controls roughly one-third of the drainage area. The Rubicon joins the Middle Fork about 25 miles upstream from the proposed dam site. The North Fork and Middle Fork converge just above the dam site. Mean annual precipitation over the basin is 58 inches.



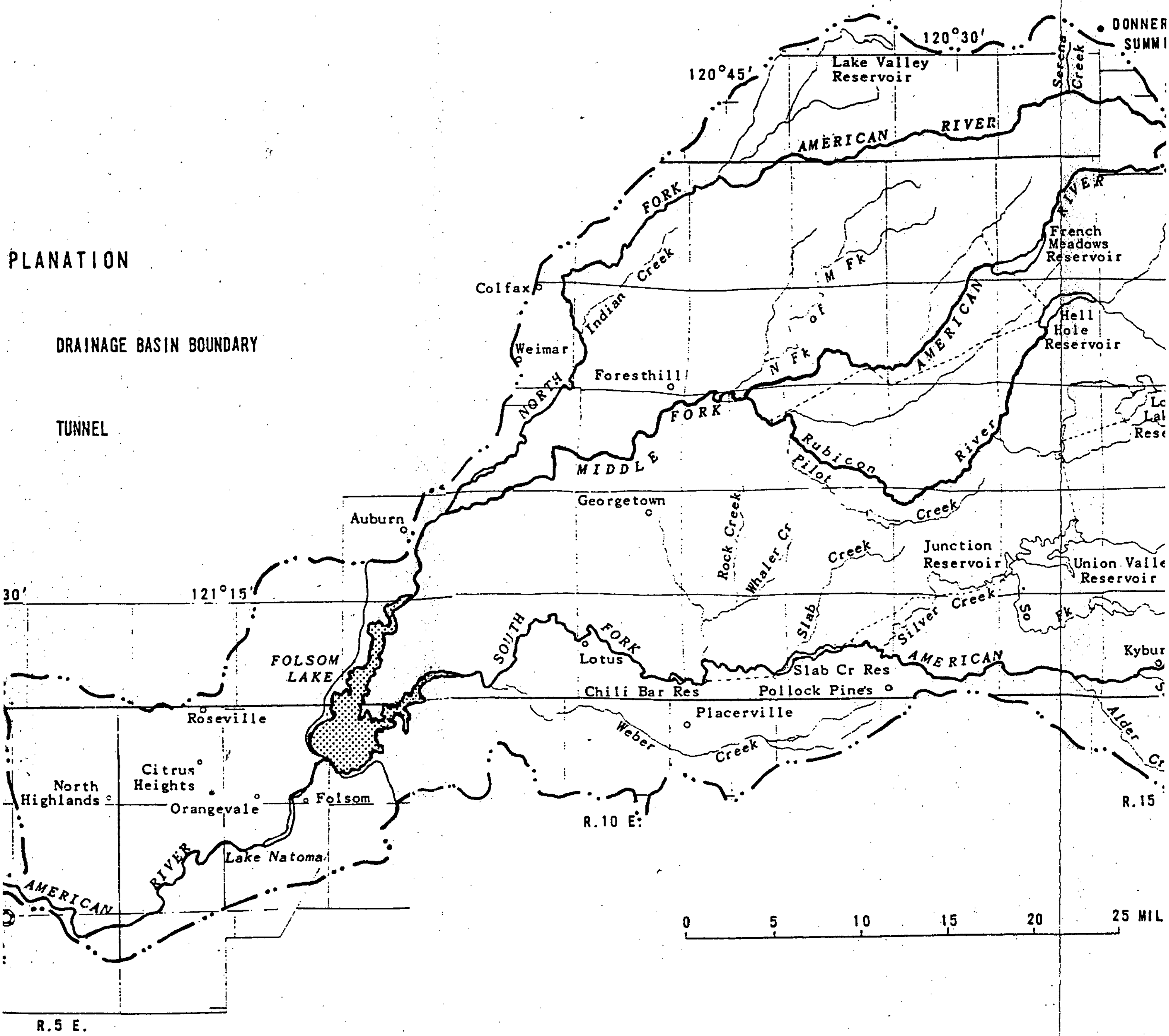
Longitudinal Profile of the Rubicon and Middle Fork Stream Channels

Source: USGS, 1968

FIGURE 4.10-1


EXPLANATION
DRAINAGE BASIN BOUNDARY
TUNNEL
AREA OF THIS REPORT
CALIFORNIA
Sacramento R
SIERRA NEVADA
Auburn
121°30′
121°15′
FOLSOM LAKE
38° 45′
Roseville
T.10 N.
SACRAMENTO
Citrus Heights
North Highlands
Orangevale
Folsom
AMERICAN
RIVER
Lake Natoma
SACRAMENTO
RIVER
38° 30′
R.5 E.

America



American River Drainage Basin

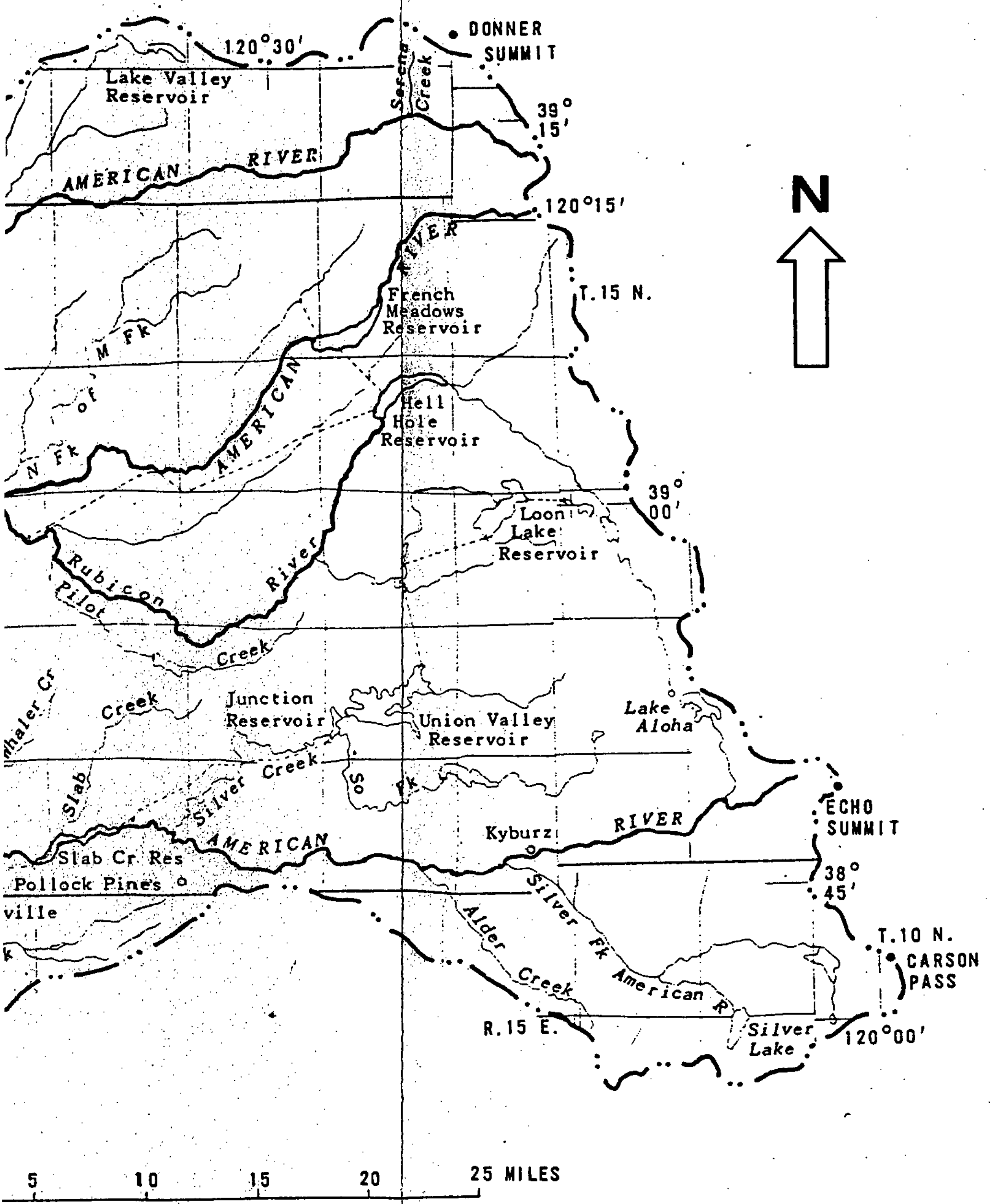


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FIGURE 4.10-2

Topography throughout the basin is steep. The upper part of the basin is quite rugged with a large amount of exposed rock. Soil cover in the lower elevations is a reddish silty clay. Vegetation varies considerably in both density and composition because of the large elevation range. Main types of vegetative cover are intergradations of oak woodland, chaparral, and conifer forest. Timber density varies from moderately heavy to light. Logging operations are present.

The area contributing sediment to the gravel extraction site is approximately 614 square miles (USGS, 1979). The drainage area contains numerous lakes and reservoirs. Table 4.10-2 lists these impoundments. These reservoirs have some reducing effect on sediment inflow; however, sediment yield studies, in an effort to maintain a conservative analytical approach, assumed a zero trap efficiency for all structures within the drainage basin (USBR, 1967). The net effect is an overestimate of sediment transport. However, the presence of the structures has implications for gravel recruitment because much of the coarse alluvium transported by the river originates in the upper portions of the drainage basin where the reservoirs are located and the tributary gradient is the highest.

River Flow

The Middle Fork stream flow is variable. Generally, the highest flow rate occurs during the spring months. Mean annual precipitation averages about 60 inches in the upper elevations of the drainage basin. After the winter and spring flows have subsided, stream flow consists of power releases from utility company reservoirs and minimum required releases. Figure 4.10-3 is a schematic diagram showing diversions and storage in the Middle Fork and Rubicon River basins.

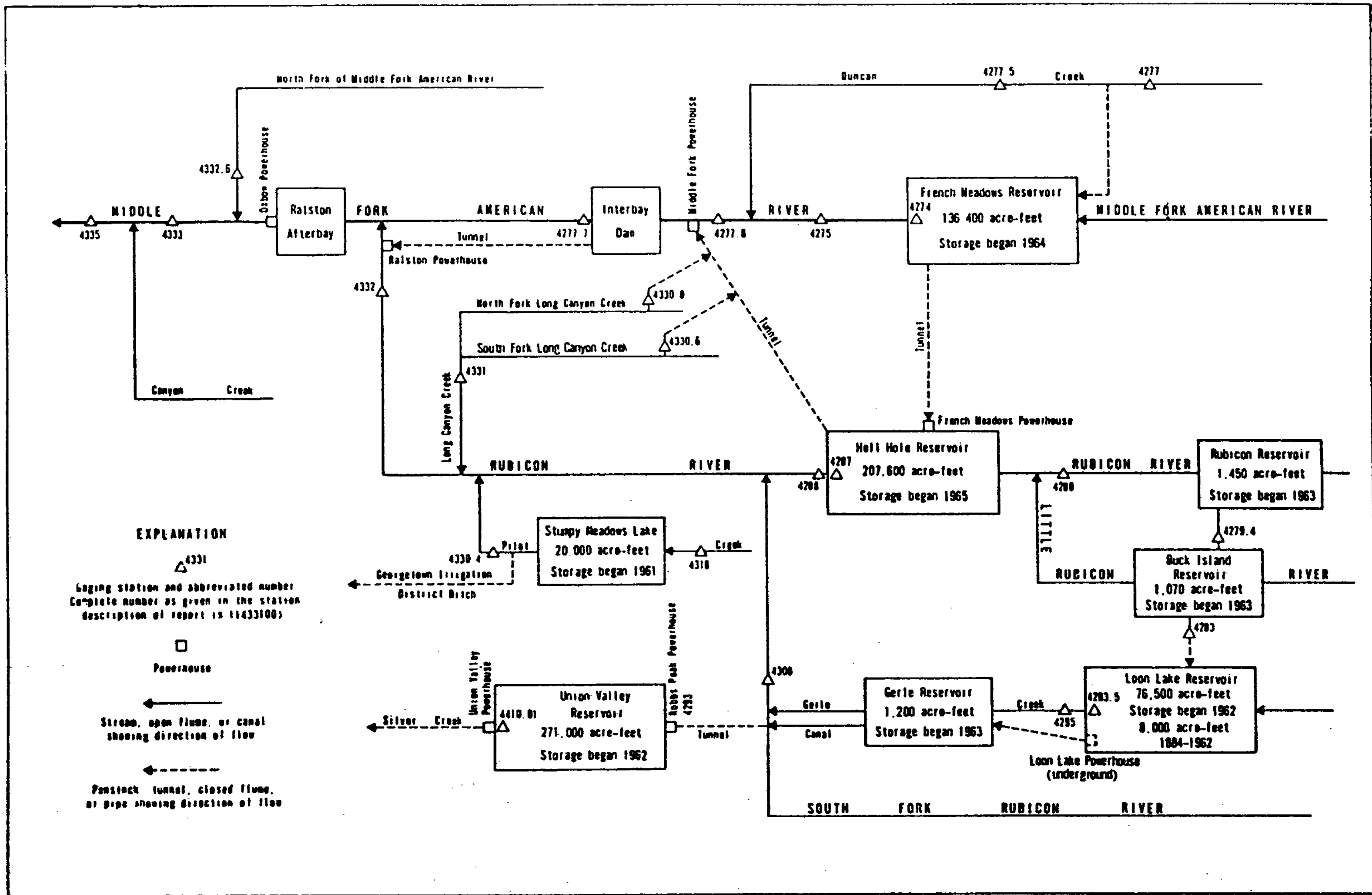
Upstream power generation reservoirs provide no flood protection. Overbank flow occurs during winter and spring months when the river reaches flood stage. Examination of Flood Insurance Rate Maps of the area indicates the 100-year flood plain roughly corresponds to the lateral extent of the gravel bars. This suggests that portions of the bars are inundated during greater frequency events. High water marks observed at the bars indicate recent water surface elevations of as much as 10 feet above bank height occur during flood stage. Severe floods have inundated areas outside the current floodplain as evidenced by USGS discharge data (USGS, 1985).

Discharge rates through the proposed mining area on the Middle Fork have been recorded by a stream gauge located 1.4 miles upstream of the confluence with the North Fork (USGS gauge no. 11433500, see Figure 4.10-1). Historic data (74 years) show an average discharge rate of 1,300 cubic feet/second (cfs). The average annual maximum discharge is about 5,000 cfs, and the average annual minimum is about 100 cfs. The maximum discharge for the period of record, 253,000 cfs, occurred as a result of the Dec. 23, 1964 failure of the partially completed Hell Hole Dam on the Rubicon River (Figure 4.10-4).

TABLE 4.10-2
MAJOR RESERVOIRS - MIDDLE FORK AMERICAN RIVER BASIN

Reservoirs	Stream[1]/ Tributary	Owner[2]	Elev. Top of Dam (ft.)	Storage (acre-feet)
L.L. Anderson (French Meadows)	M.F.	PCWA	5,271	133,700[3]
Hell Hole	Rubicon River		4,650	208,400
Lake Edson (Stumpy Meadows)	Pilot Creek	GDPUD	4,272	20,000
Loon Lake	Gerle Creek	SMUD	6,418	76,500
Ralston Afterbay	Rubicon River	PCWA	1,189	850
Rubicon Springs	M.F.	SMUD	6,251	1,450
Oxbow	M.F.	PCWA	---	2,800

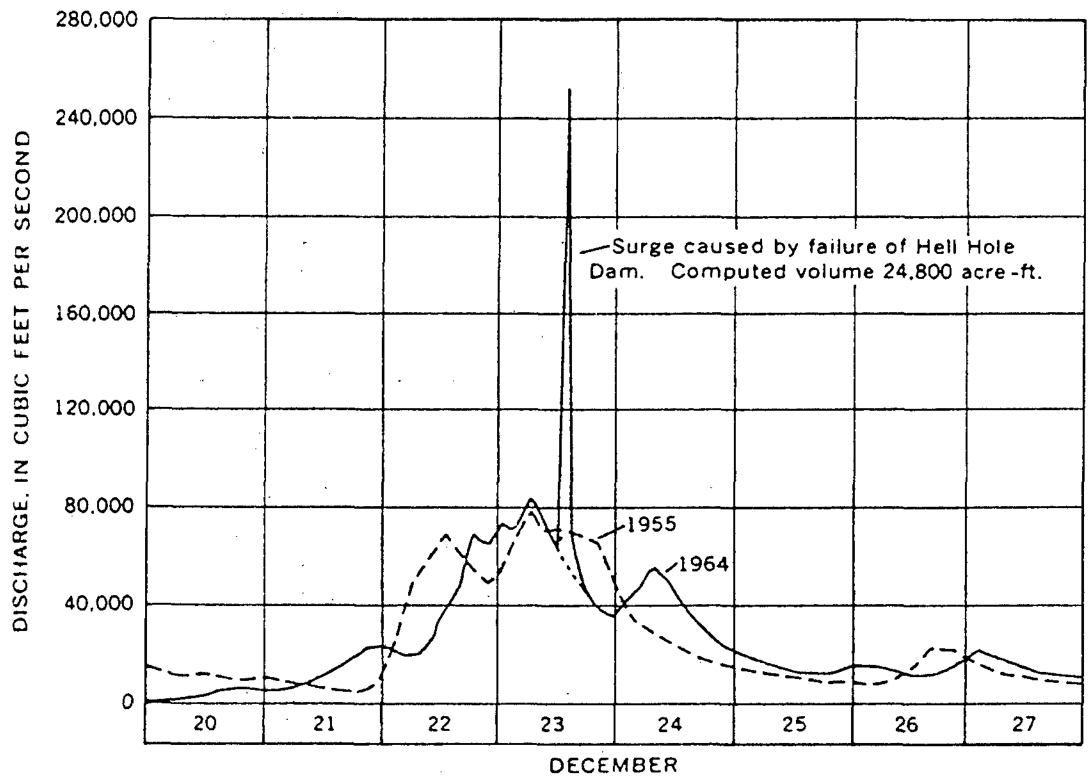
[1] M.F. - Middle Fork American
[2] PCWA - Placer County Water Agency
GDPUD - Georgetown Divide Public Utility District
SMUD - Sacramento Municipal Utility District
[3] Effective storage is reduced during winter months for dam safety.



Schematic Diagram Showing Diversions and Storage in Middle Fork American and Rubicon River Basins

Source: USGS, 1985

FIGURE 4.10-3



Hydrographs at Middle Fork American River Near Auburn

Source: USGS, 1968

FIGURE 4.10-4

Studies by the U.S. Geological Survey documented the effects of the Hell Hole flood event (Scott and Gravlee, 1968). The surge release produced peak discharges substantially in excess of previously recorded flows for which there are data. The discharge was 3.3 times the magnitude of the 100-year flood on the Middle Fork, 36 miles downstream from the reservoir. Average velocity of the flood wave was 22 feet per second. Erosion of materials from steep, thickly mantled canyon walls resulted in thalweg aggradation at five measured profile sites. Stripping of the lower valley side slopes may have triggered a period of increased mass movement in the gorge of the Rubicon River.

Terrace-like boulder berms, probably associated with macroturbulent transport of boulders in suspension, formed in backwater areas in the uppermost Rubicon River canyon. Boulders were piled to a depth of 5 feet on a terrace 28 feet above the thalweg at a peak stage of 45 feet. Boulder fronts as much as 7 feet high formed lobate scarps transverse to the channel indicating bed material moved as viscous subaqueous rockflows. Movement of coarse detritus in large gravel waves may also have occurred.

Sediment Yield

Sediment transported by a river can be divided into three fractions: 1) dissolved load, 2) suspended load and 3) bed load. The compounds in solution or colloidal mixtures are the dissolved load. The solid matter is either fine-grained particles in suspension (suspended load) or coarse-grained particles (sand and gravel) that slide, roll or bounce along the stream bed (bed load). Division of the load varies greatly among rivers and is controlled by climatic and structural factors. Both dissolved and suspended loads in river water are routinely measured at gaging stations. Bed load defies attempts to measure it accurately due to physical conditions surrounding its transport. A common practice is to estimate bed load as a percentage of suspended load.

During the design stages of the original multi-purpose Auburn dam, the USBR estimated the amount of storage within the reservoir lost to sedimentation (USBR, 1967). Suspended sediment data from a stream gauge near the dam site was used to calculate sediment yield rates for the 974-square-mile drainage basin. A total sediment yield rate of 0.27 acre-feet/square mile/year was computed using the flow duration-sediment rating curve method (USBR, 1967). Since no bed load data were available, a value of 25 percent of suspended sediment load was used for the estimated bed load based upon observation of the channel and earlier sediment studies.

From the results of the USBR calculations, the amount of gravel replenishment at the extraction site can be estimated. Applying the 25 percent bed load correction factor to the total sediment yield rate of 0.27 acre-feet/square mile/year, and multiplying by the drainage area contributing to flow through the site (614 square miles), results in a gravel replenishment rate of 53,000 cubic yards per year.

4.10.2 IMPACT ANALYSIS

The total quantity of material available from the Middle Fork American River bars and the river channel is about 9.6 million cubic yards. If the river channel material is ignored, the total quantity of material from the bars alone is 8.6 million cubic yards, or about 27 percent more material than the quantity required for the dam. Because much of the material is too fine for use as concrete aggregate, all of the material in the bars would be required. Therefore, it is assumed that extraction of all material contained in the bars would be necessary to meet the demands for dam construction and that the one million yards beneath the river channel would be left in place.

Sediment Transport Rates

The theoretical maximum amount or mass of sediment that a stream can transport is called its capacity. The grain size of the detritus may partly determine how much can be carried, but capacity is primarily a measure of the maximum amount, not grain size, of the load. Competence, on the other hand, is the measure of a stream's ability to transport a certain maximum grain size of sediment. Competence depends primarily on velocity, although channel shape, the shape and degree of sorting of the sediment particles, amount of suspended load, and water temperature can also affect competence.

It can be stated generally that streams do most of their transporting while in flood, and most of their depositing when the floods recede. In general, the coarse material transported by a stream moves intermittently and much of the coarsest material may be at rest for all but brief periods of time. Other things being equal, the length of time between moves and the distance moved is a function of particle size.

A salient issue regarding the Middle Fork sand and gravel mining proposal is whether or not the alluvial deposits, once removed, will be replenished within a relatively short time frame. The gravel bars now located along the river are continuously being eroded and reformed by fractional amounts. Because they require the least energy to keep in suspension, most of this continuous transport is of smaller-sized particles (sand, silt and clay).

As evidenced by the failure of the partially constructed Hell Hole Dam, it is only during large, great magnitude flooding events that the competence of the Middle Fork is adequate to move significant quantities of coarse-grained alluvium. Such events are infrequent; recurrence intervals may be on the order of hundreds or even thousands of years. The majority of the bar material was probably transported during a period of geologic history when climatic factors provided conditions conducive to rapid erosion and transport of coarse-grained material from the upper portions of the Middle Fork's drainage basin.

Climatic conditions are different today. Today's climate and present conditions may have little or no resemblance to those which controlled the formation of the Middle Fork bars.

Historic steam gauge records show an average annual discharge of 1,300 cfs over the last 74 years. Average highs range to around 5,000 cfs. The Hell Hole disaster, during which large quantities of coarse material were transported miles downstream, produced a peak flow 50 times greater than the average annual highs. From this evidence it can be inferred that the flow required to transport significant quantities of coarse alluvium does not occur frequently enough to ensure timely replenishment of the gravel bars.

Additional insight into gravel recruitment potential can be gained by examining sedimentation rates. The sedimentation study conducted by the USBR estimated the sediment yield of the drainage basin to be 0.27 acre-feet/square mile/year. From this figure, a bed load transport rate of 53,000 cubic yards/year was calculated based on assumptions detailed above (see Sediment Yield). Given that project implementation would result in removal of 8.6 million cubic yards, the calculated replenishment rate amounts to less than one percent per year. In reality, the replenishment would probably be much less due to the effects of the various upstream impoundment facilities and the large bed load fraction assumption.

<u>Changes in Channel Morphology</u>

Depths to bedrock beneath the gravel bars range from less than 10 feet at the upper bars to 90 feet at the lower bars. The average depth of material in the bars ranges from 18 to 31 feet with an overall average depth of 26.5 feet. Depth to bedrock beneath the stream channel is approximately 10 feet.

Full implementation of this alternative would require removal of approximately 90 percent of the alluvial material contained within the bars. Initially, what alluvial material there was left within and adjacent the stream channel would be redistributed to a pre-mining configuration; that is a series of lateral and point bars redistributed over the 7-mile reach.

However, the bars would be reduced to the extent that they would no longer constitute the significant geomorphological feature they now are. Much of the residual alluvium left by the mining operation would consist of boulders and oversize cobbles too large to process economically. Ultimately, the stream would likely form a boulder strewn bedrock channel, similar to Ruck-A-Chucky Rapids upstream from Cherokee Bar. The relatively slow gravel replenishment rate precludes the possibility of a rapid return to a pre-mining configuration.

Ultimately, the operation would lower the streambed. Lowering of the river bed by sand and gravel extraction will cause a localized increase in longitudinal slope at the upstream end of a given excavation. This will increase the velocity of the flow in the mined reach and the sediment carrying capacity of the river. The higher erosive capacity may cause erosion of the river bed and/or banks at both upstream and downstream edges of the mined areas.

Higher erosive capacity in the region of extraction could also result from the reduced sediment load along the affected reach. Removal of sand and gravel from the river system

would decrease the sediment available for transport. Water that once used its energy to transport sediment would have less sediment to carry, and thus would be laden below its transport capacity. The excess energy could then be free to act as an erosive force.

Erosion can also be caused by changes in flow pattern or channel alignment during excavation. Reaction of the river system to changes in slope, sediment load and flow direction will to some extent depend on the relative resistance of river bed and bank material. The most easily eroded material will be acted on first. Excavation of the river bed and bank material, and improper final channel shaping can redirect the force of the river's flow against erodible banks. Implementation of this aggregate alternative would result in significant morphological impacts to the Middle Fork river channel (Class I). The large quantities of materials involved would preclude conventional mitigations such as cross-sectional and longitudinal grade control.

4.10.3 MITIGATION MEASURES

Complete removal of the 10 sand and gravel bars along the Middle Fork would result in significant unavoidable impacts to stream channel morphology with corresponding implications for recreational, visual and biological resources. Unless major climatic changes occurred, little or no gravel recruitment would occur in the near future; the changes would be permanent within the context of a human time-scale. Little or no mitigation exists which would effectively replace the site-specific loss of resource or prevent potential erosion and flooding impacts.

During the 1960s and 1970s when project implementation involved construction of a multi-purpose dam with a permanent pool, exploitation of the aggregate resource along the Middle Fork would have had less impact. However, since the current flood control proposal would entail only periodic inundation of the canyon bottom, removal of the bars would represent significant morphological changes to an environment which supports a variety of resources.

Current aggregate harvesting techniques involve removal of only so much material as can be replaced on an annual basis. A sound program of gravel mining management specifies a regulated gravel extraction rate based on replenishment rates. Because of the large quantities of aggregate necessary to implement the current proposal, techniques such as bar skimming are not a viable recourse toward impact minimization.

5.0 REPORT PREPARERS

This report was prepared by the Sacramento office of Fugro-McClelland (West), Inc. under contract to the Corps of Engineers and the Department of Water Resources. Individuals contributing to report preparation included:

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6.1 PERSONS CONTACTED

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AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX M

CHAPTER 11

EXISTING LEVEE BREACH DETERMINATIONS

MAY 1990

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

EXISTING LEVEE BREACH DETERMINATIONS

This report describes the levee breaching scenario developed for the levees in the Sacramento Area. This scenario was based upon engineering studies and recommendations by different engineering disciplines as well as taking into account historical flood elevations and recent criteria to be used in determining breach elevations for existing levees. One primary assumption used in this evaluation is that any levee repairs identified in the Sacramento River Flood Control System Evaluation Initial Appraisal Report, Sacramento Urban Area, have been completed.

Engineering analysis considered five major A/E and in-house levee stability studies. Other factors given weight in the engineering evaluation included wind and wave actions, flow velocities, duration of high stages, and erosion potential of the levee material. The existing conditions of the levees were also evaluated with respect to animal borings, cracks and homogeneity, and woody vegetation on or near levees. Additional information used in the analysis came from monitoring maintenance inspection records, determining locations of historical seepage and failure problems, experience from developing emergency repair plans, and levee performance under reoccurring flood stages. All of these factors were considered in developing an engineering determination of when levees might fail.

Further guidance on failure scenarios states that no levee should be assumed to fail if it has historically withstood higher elevations than might be indicated from a breaching scenario developed strictly from an engineering standpoint. Also, for existing levees, the initial failure scenario should consider encroachment into one-half the current design freeboard. Guidance suggests that in addition to taking into account the engineering evaluations, the final scenario should reflect the historic elevation a levee has sustained or one-half freeboard encroachment, whichever is greater, unless there are overriding reasons not to.

This guidance was applied to the Sacramento Area levees. The Engineering Evaluation criteria was applied to the system and initial failure reaches identified. Six failure points were identified and are shown on Plate 1. Four failure reaches are along the American River and two in the RD 1000 levees surrounding Natomas. The six reaches are plotted along with the different failure criteria and are shown on Figures 1-6. In these figures, the Breach Elevation profile represents the initial engineering evaluation of when the levees would fail, the 1/2 Freeboard profile is encroachment into one-half the design freeboard, and the 1986 High Water Mark profile represents the historical high water elevation on the levee. Final determinations for each reach are discussed below and shown on Table M-11-1.

* American River Break 1 - Figure 1 - The one-half the freeboard criteria is higher than the engineering evaluation for a portion of this reach. However, a stability analysis of the American River, Appendix M, Chapter 2, determined that this reach of levee begins to exhibit stability problems when water elevations are five feet from the levee crown. For this reason, the engineering evaluation for levee failure for this reach was retained.

* American River Break 2 - Figure 2 - The one-half the freeboard criteria is slightly higher than the engineering evaluation breach elevation for most of this reach. However, the American River stability analysis mentioned above determined that this reach of levee begins to exhibit stability problems when water elevations are six feet from the levee crown. Also, velocities become erosive at high flows in this reach. For these reasons, it failure would very likely occur at four feet from the top and the engineering evaluation for levee failure in this reach was retained.

* American River Break 3 - Figure 3 - For this reach the engineering evaluation breach elevation is higher than both the historical high water and the one-half freeboard criteria. In addition, the American River stability analysis determined that this reach of levee begins to exhibit stability problems when water elevations are six feet from the levee crown and velocities become erosive at high flows in this reach. Again using the stability analysis, the levees would most probably fail at elevations five feet from the top of levee and the engineering evaluation for levee failure for this reach was retained.

* American River Break 4 - Figure 4 - The engineering evaluation of levee failure at four feet from the top was retained for this reach because of levee instability problems similar to break 2.

* Natomas East Main Drain - Figure 5 - For this break point, the historical high water and one-half freeboard criteria were both higher than the 3 foot criteria selected from the engineering evaluation. Because of this the breaching criteria was changed to be 1.5 feet from levee top. For this figure, the Breach Elevation profile represents the 1.5 foot failure criteria.

* Natomas Cross Canal - Figure 6 - For this break point the engineering evaluation was higher than the historical high water but lower than the one-half design freeboard criteria. Based on this it was decided to change the failure criteria to two feet from levee crown.

Table M-11-1 gives the adopted flood encroachments to be used in determining breach elevations and frequencies for economic analysis.

The subjective nature of any levee failure scenario developed should be recognized. The development of a levee failure scenario is fraught with difficult technical considerations and economic, social and moral aspects that could jeopardize or bias the formulation of an acceptable project. The attempt has been to determine the most probably elevation at which failure would occur. These elevations represent information to be used for economic analysis only and should not be construed as absolute failure points for determining the flood safety of any of the areas under study.

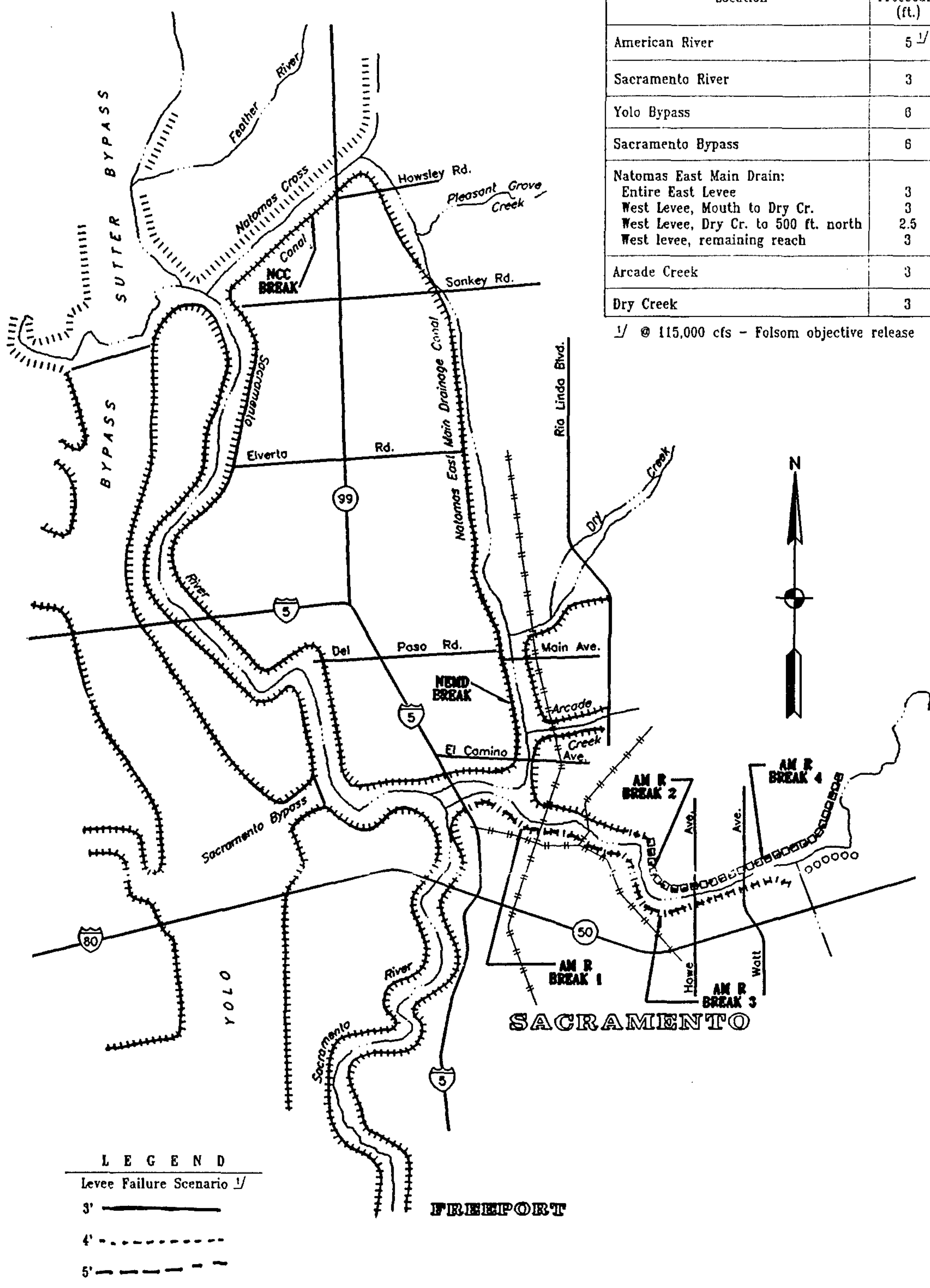
TABLE M-11-1

ADOPTED FAILURE LEVELS FOR EXISTING LEVEES USED FOR ECONOMIC PURPOSES ONLY

Levee Reach	Failure Levels From Top of Levee In Feet
1. RECLAMATION DISTRICT 1000	
a. Sacramento River (Left Bank) - Natomas Cross Canal to the Natomas East Main Drain	3
b. Natomas Cross Canal (North and South Levees)	2
c. Natomas East Main Drain - West Levee	1.5
2. AMERICAN RIVER LEVEE SYSTEM	
a. Right Bank, Sacramento River to River Mile 5.2	3
b. Right Bank, Upstream of River Mile 5.2	4
c. Left Bank, Sacramento River to River Mile 5.2	5
d. Left Bank, R.M. 5.2 to River Mile 7.8	5
e. Left Bank, Upstream of River Mile 7.8	4
3. DRY CREEK, ARCADE CREEK, AND THE EAST LEVEE OF THE NATOMAS EAST MAIN DRAIN	3
4. SACRAMENTO RIVER (LEFT BANK) FROM SACRAMENTO TO FREEPORT	3
5. SACRAMENTO RIVER (RIGHT BANK) FROM THE SACRAMENTO BYPASS TO RIVERVIEW	3
6. YOLO BYPASS AND TRIBUTARY LEVEES	3
7. SACRAMENTO RIVER (RIGHT BANK) FROM THE NATOMAS CROSS CANAL TO THE SACRAMENTO BYPASS	3

Location	Design Freeboard (ft.)
American River	5 1/
Sacramento River	3
Yolo Bypass	6
Sacramento Bypass	6
Natomas East Main Drain: Entire East Levee West Levee, Mouth to Dry Cr. West Levee, Dry Cr. to 500 ft. north West levee, remaining reach	 3 3 2.5 3
Arcade Creek	3
Dry Creek	3

1/ @ 115,000 cfs - Folsom objective release



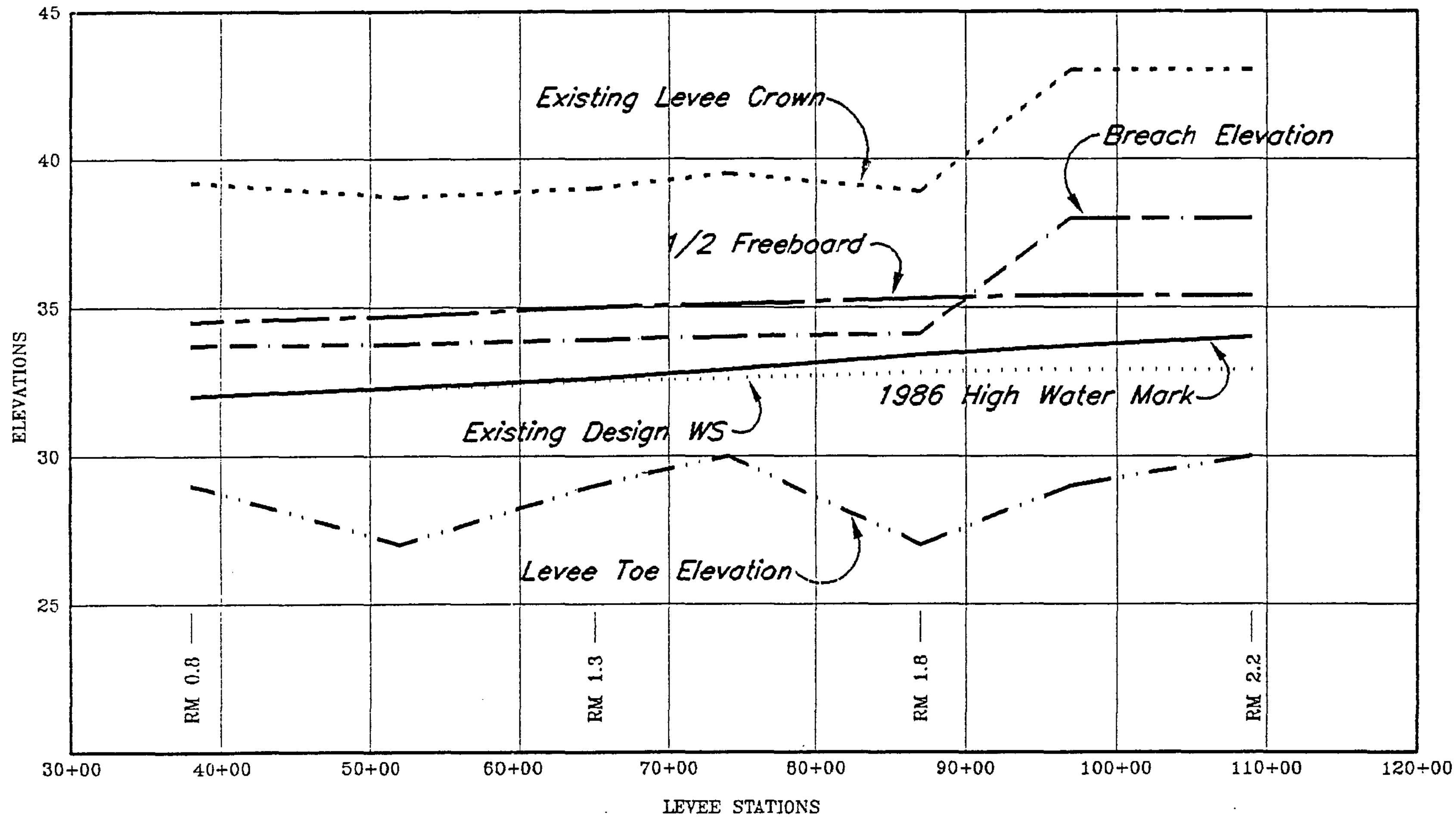
1/ Assume Failure occurs when Water is within amounts indicated below levee crown.

AMERICAN RIVER WATERSHED
CALIFORNIA

FEASIBILITY STUDIES

PLATE 1

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MAY 1990



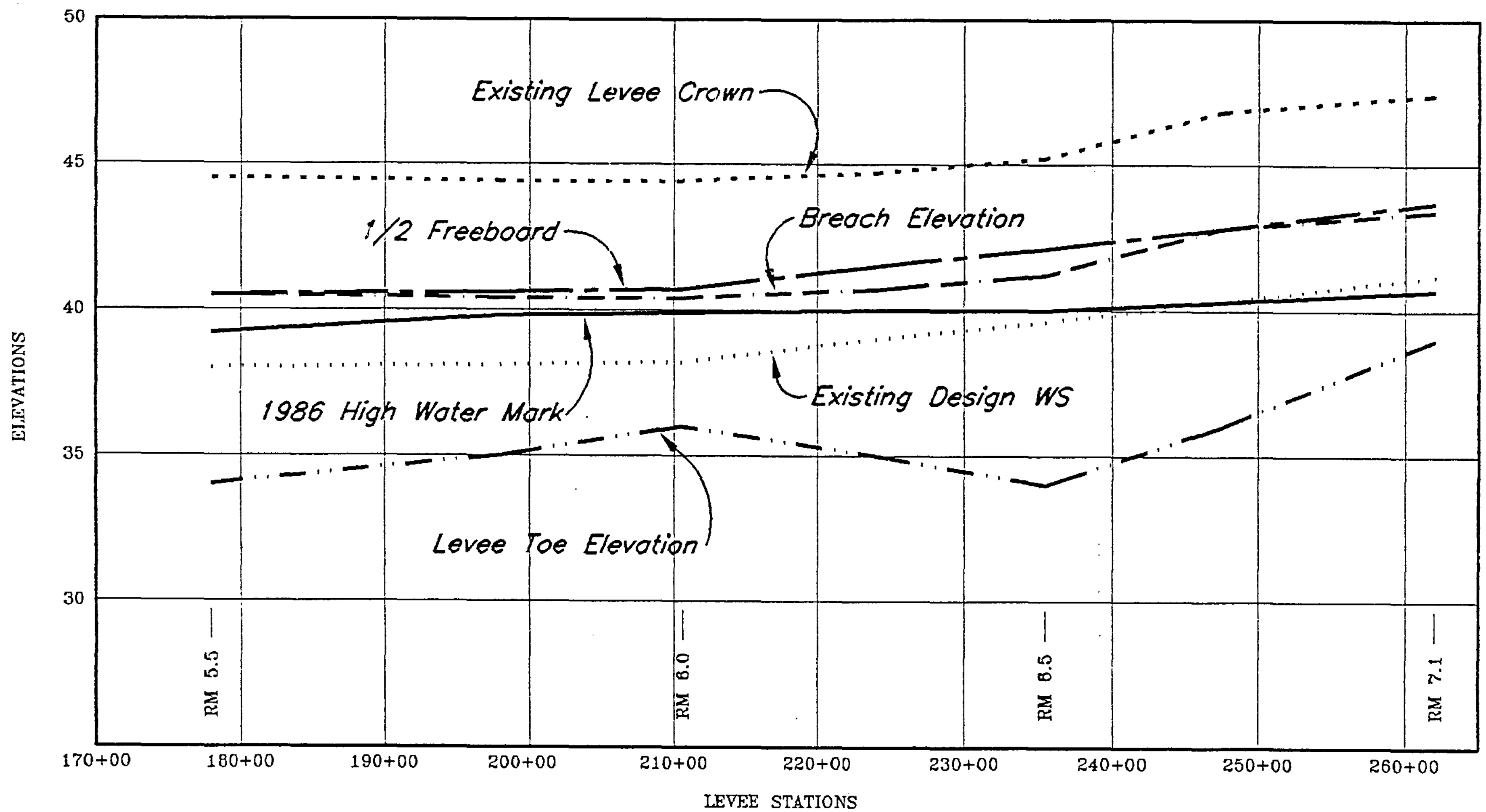
NOTE: Stability problems begin to occur at RM 1.8 when WS is within 5 feet of levee crown.

AMERICAN RIVER WATERSHED
CALIFORNIA

AMERICAN RIVER
BREAK 1
LEVEE PROFILE
SOUTH LEVEE

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MAY 1990

FIGURE 1



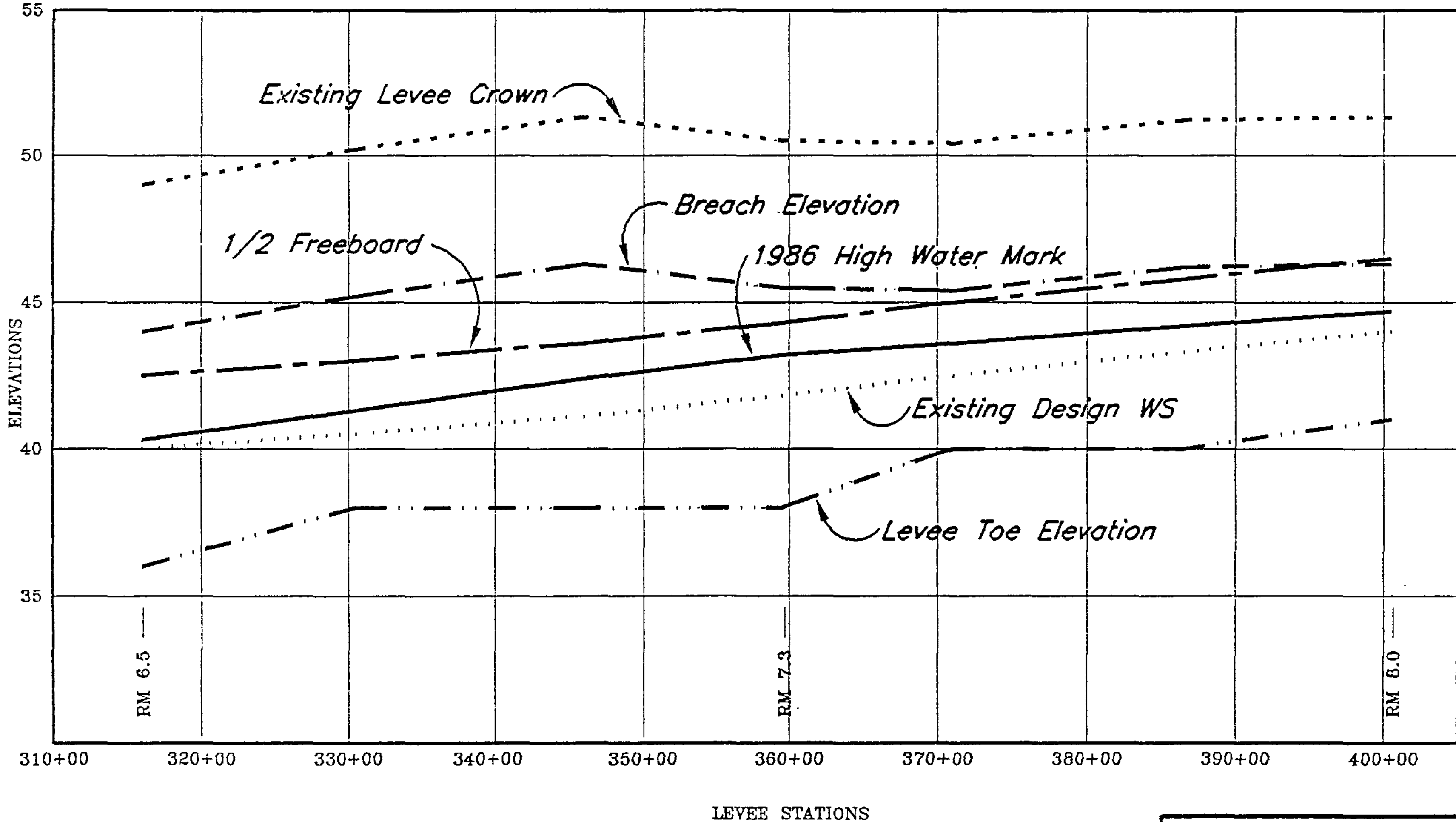
NOTE: Stability problems can occur at RM 6.5 when WS is within 6 feet of levee crown. Velocities become erosive at high flows.

AMERICAN RIVER WATERSHED
CALIFORNIA

AMERICAN RIVER
BREAK 2
LEVEE PROFILE
NORTH LEVEE

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MAY 1990

FIGURE 2



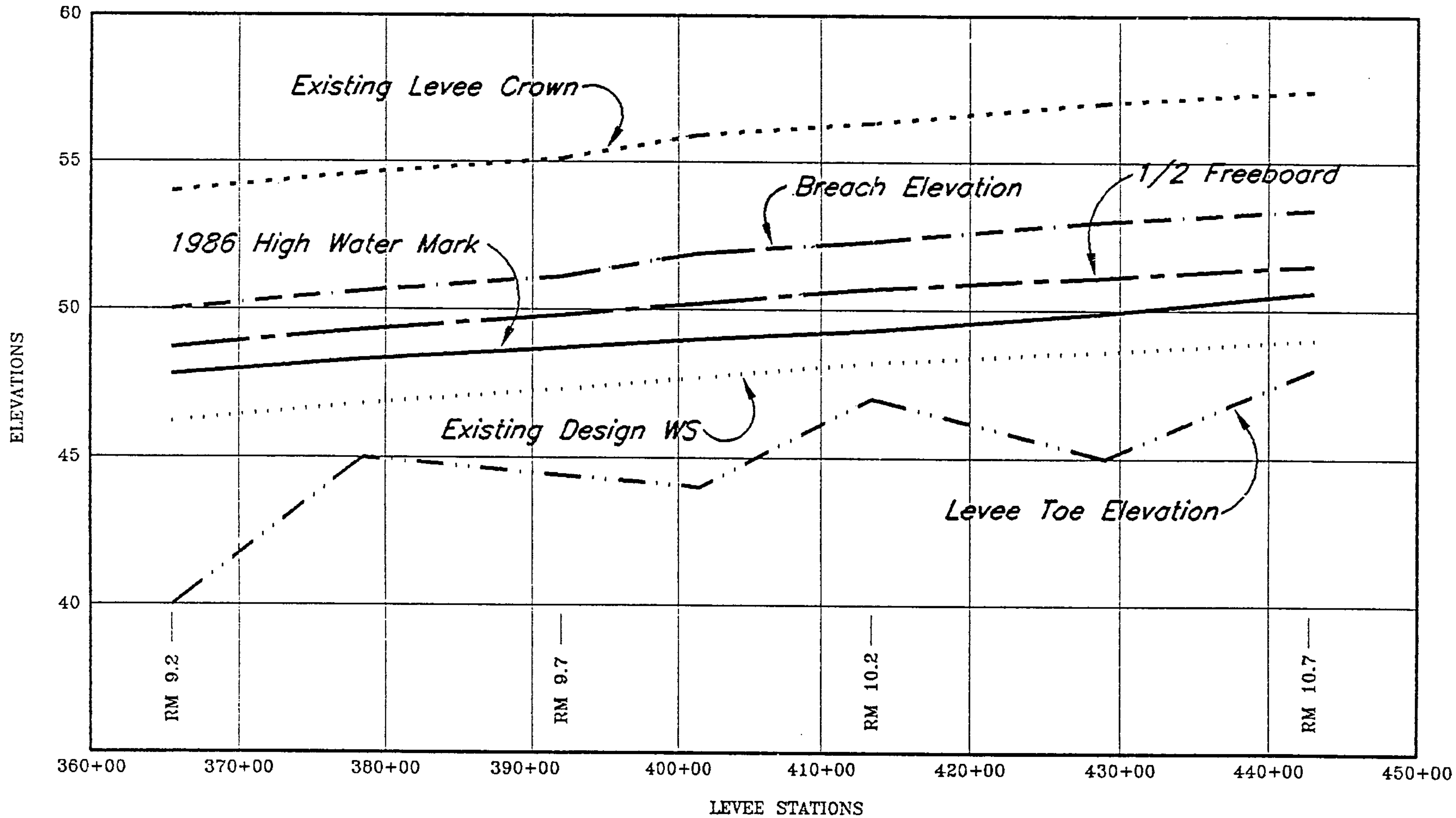
NOTE: Stability problems can occur at RM 7.0 when WS is within 6 feet of levee crown. Velocities become erosive at high flows.

AMERICAN RIVER WATERSHED
CALIFORNIA

AMERICAN RIVER
BREAK 3
LEVEE PROFILE
SOUTH LEVEE

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MAY 1990

FIGURE 3



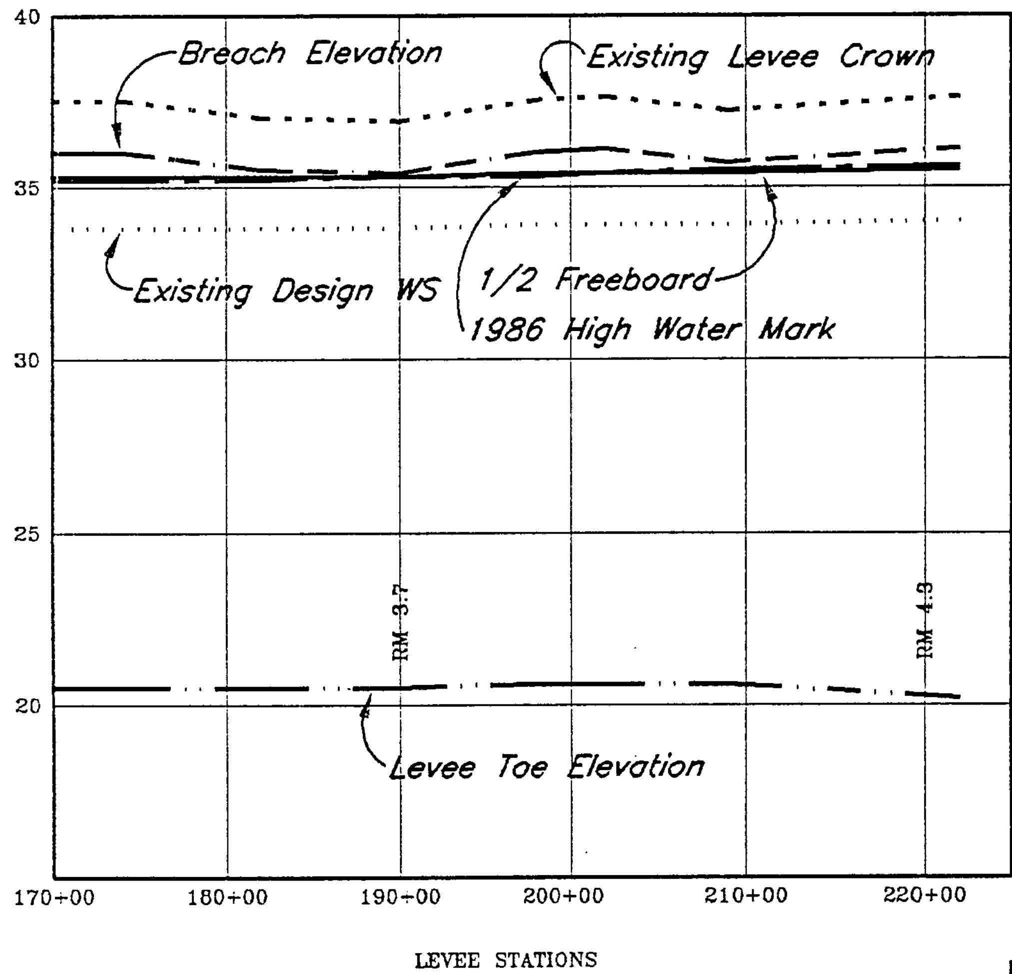
NOTE: Stability problems can occur at RM 9.2 when WS is within 5 feet of levee crown.

AMERICAN RIVER WATERSHED
CALIFORNIA

AMERICAN RIVER
BREAK 4
LEVEE PROFILE
NORTH LEVEE

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MAY 1990

FIGURE 4

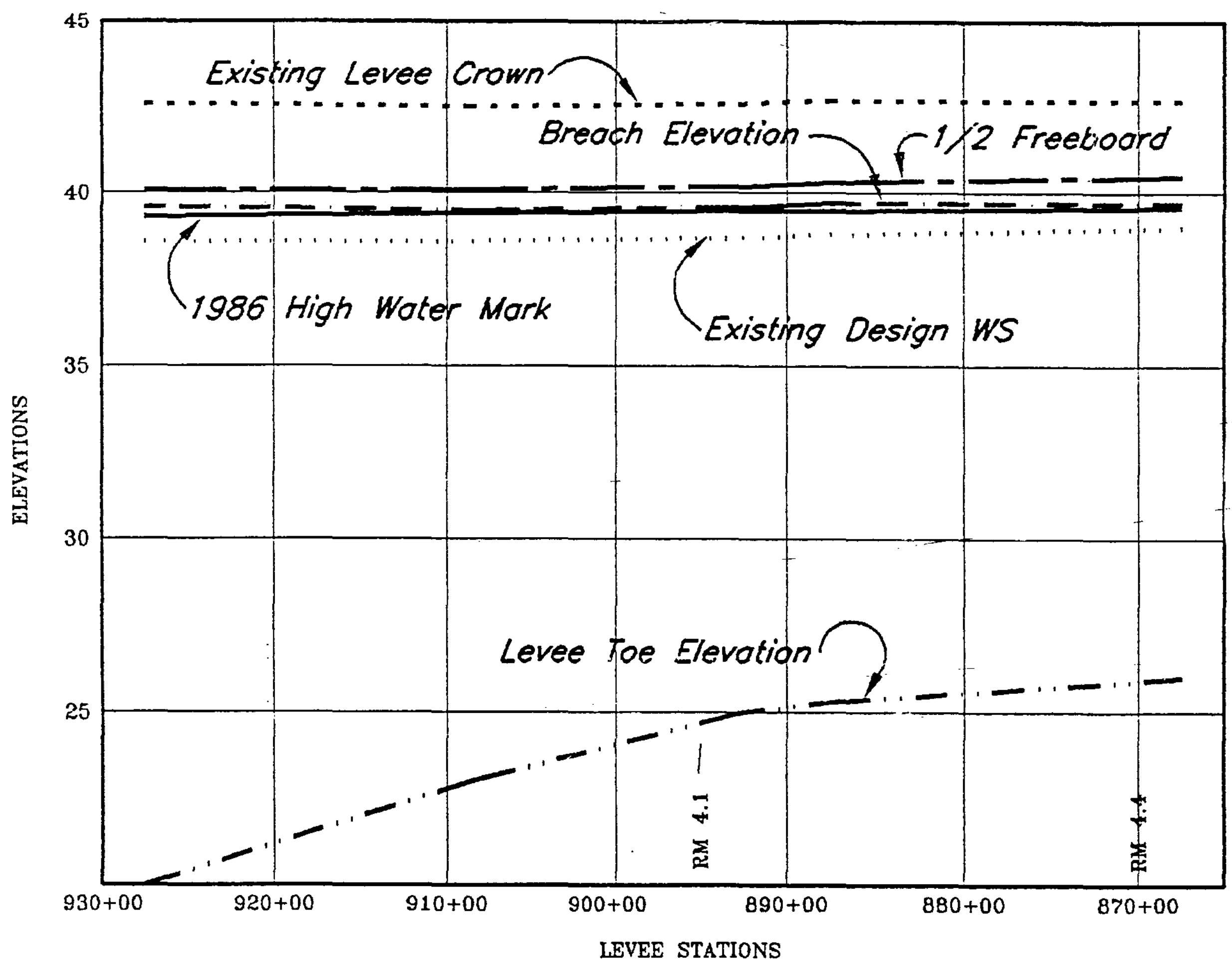


AMERICAN RIVER WATERSHED
CALIFORNIA

NATOMAS EAST MAIN DRAIN – NEMDC BREAK LEVEE PROFILE WEST LEVEE

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MAY 1990

FIGURE 5



AMERICAN RIVER WATERSHED
CALIFORNIA

NATOMAS CROSS CANAL
NCC BREAK
LEVEE PROFILE
SOUTH LEVEE

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MAY 1990

FIGURE 6